DESIGN OF WELDED STEEL STRUCTURES

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BY

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SECOND EDITION

LONDON

SIR ISAAC PITMAN & SONS LTD.

First published 1939
Second edition 1948
Reprinted 1953
Reprinted 1958
Reprinted 1962

SIR ISAAC PITMAN & SONS LTD.
PITMAN HOUSE, PARKER STREET, KINGSWAY, LONDON, W.C.2
THE PITMAN PRESS, BATH
PITMAN HOUSE, BOUVERIE STREET, CARLTON, MELBOURNE
22-25 BECKETT'S BUILDINGS, PRESIDENT STREET, JOHANNESBURG

ASSOCIATED COMPANIES
PITMAN MEDICAL PUBLISHING COMPANY LTD.
46 CHARLOTTE STREET, LONDON, W.1
PITMAN PUBLISHING CORPORATION
2 WEST 45TH STREET, NEW YORK
SIR ISAAC PITMAN & SONS (CANADA) LTD.
(INCORPORATING THE COMMERCIAL TEXT BOOK COMPANY)
PITMAN HOUSE, 381-383 CHURCH STREET, TORONTO

MADE IN GREAT BRITAIN AT THE PITMAN PRESS, BATH
F2—(T.31)

PREFACE

ALTHOUGH much has been written on metallic arc welding and the literature on the subject is growing at an overwhelming pace, most of the writing is in the nature of descriptions of work done or in reports on investigations into specific problems. There remains the want of a simple textbook in which the information is assembled in a form which will enable the engineer to make use of the process in construction in those classes of work where its advantages obviously recommend its use. An endeavour is made to place before the designer of constructional steel work the elements of design and practice of metal-arc welded construction in a simple and practical form.

The author is indebted to Mr. S. M. Reisser, B.Sc., A.M.Inst.C.E., for assistance in the preparation of the diagrams and other materials; to Mr. E. S. Needham, A.M.Inst.C.E., for reading the manuscript and making many useful suggestions; to Dr. J. H. Paterson and Murex Welding Processes, Ltd., for permission to use certain data and drawings.

A. R. M.

LONDON.

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THE DESIGN OF WELDED STEEL STRUCTURES

CHAPTER I

PARENT METAL AND WELD METAL

Parent Metal. MILD STEEL. Any present-day commercial mild steel conforming to the requirements of British Standard Specification No. 15 is suitable for welding. Steels with a carbon content of less than 0·25 per cent are not greatly affected by heat treatment, and consequently they show no significant alteration due to the heat of welding. Corrosion-resisting steels such as 0·3 per cent copper-bearing steel, etc., are readily weldable with suitable electrodes and retain their corrosion-resisting properties unimpaired.

HIGH TENSILE STEELS. All steels likely to be used for structures can be welded but when the tensile strength exceeds 37–40 tons per in.^{2} special precautions must be taken. For steels of moderate strength it is necessary, if undesirable hardening of the metal at the weld junction is to be avoided, to increase the minimum size of run in order to increase the amount of heat put into the plate, and for the higher strength steels it is necessary to heat the work before welding or to weld with special austenitic electrodes. In the case of the highest strength steels it is necessary to use austenitic electrodes and to preheat.

Guidance in the welding of high tensile steels is given in T.13—Memorandum on the Welding of Low Alloy High Tensile Steels, R.13/14—Memorandum on the Welding of High Alloy High Tensile Steels, published by the Institute of Welding.

Wrought Iron. Wrought iron is not very satisfactory material for welding because the laminations tend to tear apart under stress. A special technique is needed to obtain sound results. Iron free from laminations is welded as readily as mild steel.

Cast Steel. Provided the steel casting is of good quality, the welding is similar to that on rolled steel of the same composition.

Electrodes. Electrodes for the welding of structures are usually required to conform to the requirements of B.S. 639 for Class A electrodes, under which a test piece built entirely of weld metal shall have an ultimate tensile strength of 28 tons per in.^{2} and an elongation of 20 per cent on four diameters, and give an izod value of 30 ft. on a standard 10 mm. test piece. These properties are sufficiently close to those of structural mild steel for the weld metal to be considered as equal to the parent metal, and to be designed to carry the same allowable loads.

It is useful, however, to consider how variations in the properties of the weld metal affect the design of the welded joint or structure. The development of electrodes has gone on progressively during the past 30 years, but there are several recognizable phases in the improvement of the physical properties of the weld metal corresponding with the introduction of different classes of electrode coatings. Roughly there have been four main classes of electrodes for welding mild steel. (1) bare wire, (2) dipped or sprayed electrodes, (3) fluxed or covered electrodes, (4) covered alloy and shielded arc electrodes. The weld metal from the bare wire electrodes is subject to severe atmospheric contamination, and the physical properties are considerably inferior to those of mild steel. With the dipped or sprayed electrodes the covering as a rule acts merely as a flame producer, making the arc steadier and easier to control, and has little effect on the physical properties of the weld metal. With the fluxed or covered electrodes the running properties are further improved,

and the deoxidizing and cleansing action of the ingredients of the covering effects a substantial improvement in the physical properties of the weld metal, mainly by the elimination of impurities from the weld. In the alloy type covered electrode the addition of small quantities of various alloying materials results in almost entire elimination of impurities and produces a weld metal having physical properties very similar to those of rolled mild steel. The deposited metal is not really an alloy but a true steel. The small quantities of alloying metals, such as manganese and titanium, assist in the production of a metal of great purity and density. With the shielded arc type, a carbonaceous covering on the electrode provides a protective sheath of gas round the arc zone during welding and results in the deposition of a very pure metal of excellent physical properties.

The function of a flux covering on an electrode is: (1) to protect the weld metal from attack by the atmosphere during its passage across the arc, and to remove metallic oxide from the weld zone; (2) to prevent attack by the atmosphere on the hot deposited metal by covering it with a glass-like film of impenetrable slag; (3) to cleanse the surfaces of the metal being welded; (4) to steady the arc and make the welding operation easier; and (5) to introduce alloying elements into the weld metal if desired.

A good flux possesses the following characteristics—

It has a melting-point slightly above that of the core wire.

It forms a slag which when molten is moderately but not too fluid, and which has the right viscosity so that it covers the deposited metal uniformly and pulls away progressively from the welding zone to allow the operator to see where he is working. It does not run in front of the electrode.

It forms a brittle and non-adherent slag when cooled which is readily removed by chipping or brushing.

The flux coating must be concentric with the wire. This is the more important because the directional control of the arc depends on the covering burning back at a uniform rate all

round. The coating must be uniform in composition and must contain no harmful impurities such as sulphur, phosphorus, nitrogen, or excess water vapour. The electrodes should not emit any poisonous or noxious fumes, and the covering should not cause "growths" on the weld metal.

Physical Properties of Weld Metal. The physical properties of the weld metal deposited by typical electrodes of the four classes are given in Table I, the corresponding figures for mild steel being given for comparison.

It will be seen that there is a progressive improvement in all the properties through the four classes. The figures given in the table are average figures for typical electrodes of each class. It is possible to obtain a weld of 28 tons per in.^{2} from a bare wire

TABLE I

Comparison of Properties of Weld Metal from Different Types of Electrodes

	Bare Wire Electrodes	Dipped or Sprayed Electrodes	Fluxed or Covered Electrodes	Alloy Covered and Shielded Arc Electrodes	Mild Steel Plate
Ultimate Strength: tons per in.^{2}	18–22	22–24	24–26	28–32	28–32
Yield Point: tons per in.^{2}	Indefinite	Indefinite	16–18	20–24	18
Elongation: % on 3·54 dia.	3–6	6–10	10–15	18–25	20–30
Izod (10 mm. specimen), ft. lb.	2–5	6–10	15–25	35–70	25–50
Fatigue Limit: tons per in.^{2} $\pm$	5·6	6	8·5	11·5	12

weld with as much as 10 per cent elongation, but such results are not obtained under ordinary working conditions.

In the case of the bare wire and lightly fluxed electrode welds the metal can hardly be said to have a yield point. The stress-strain curve is similar to that of cast iron. The limit of

proportionality is low and the ratio of strain to stress increases continuously until fracture occurs. A test piece built entirely of weld metal breaks with a square fracture similar to that of cast iron, without necking and with no appreciable reduction of area.

With the fluxed or covered electrode there is a definite yield point, which may be picked up in a tensile test on all weld metal specimens by the dropping of the beam, but the horizontal portion of the stress-strain curve representing the period of elongation without increase of load is short compared with that for a mild steel specimen. There is a fair reduction of area and some necking at the break.

The behaviour of the weld metal from the alloy type and

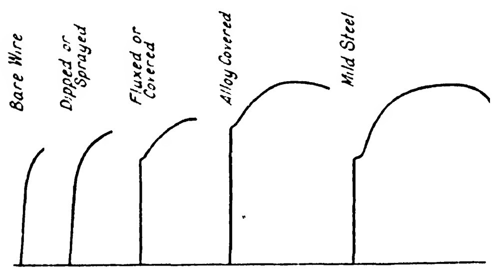


FIG. 1. COMPARATIVE STRESS-STRAIN DIAGRAMS FOR ALL-WELD-METAL TENSILE SPECIMENS MADE FROM DIFFERENT TYPES OF ELECTRODES

shielded arc type of electrode is substantially similar to that of mild steel, showing a definite yield point, large reduction of area, and considerable necking adjacent to the fracture. In Fig. 1 the shape of the stress-strain curves is given for the four types of electrode and for mild steel.

The difference in the physical properties of the weld from covered electrodes and the alloy type electrodes is more important than may appear at first sight. A difference of 2 to 4 tons per in.^{2} in the tensile strength of a steel may not be very important, but a difference of 4 tons between the tensile strength and more particularly between the yield points of the weld metal and parent metal in a joint may not be ignored. In

Table I the figure for yield point for mild steel is given as 18 tons per in.^{2}, and the lower figures for the covered and alloy type electrodes are given as 16 tons per in.^{2} and 20 tons per in.^{2} respectively, the one below, the other above the yield point of mild steel. The effects of these differences in yield point are shown very clearly by making bend tests on butt-welded specimens. (Fig. 2.) In the case of the ordinary covered electrode the weld metal starts to elongate at 16 tons per in.^{2}, before the parent metal reaches its yield point at 18 tons

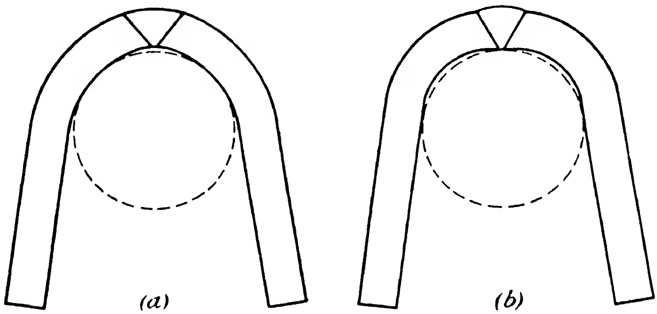


FIG. 2. BEND TEST SPECIMENS SHOWING EFFECT OF DIFFERENCE BETWEEN YIELD POINT OF WELD METAL AND PARENT METAL

per in.^{2}, and consequently the greater part of the elongation takes place in the weld. The plates are bent about the weld on a small radius and the weld may fail before any substantial elongation occurs in the parent metal. If the weld metal is very ductile, the piece may bend through a considerable angle, but if the ductility is low, fracture will occur at a small angle of bend.

When the weld is made with the high-grade covered electrode, the plate starts to elongate first when the stress on the outer fibres reaches the yield point at 18 tons, but the weld does not start to stretch until a stress of 20 tons per in.^{2} is developed. The greater part of the elongation accordingly takes place in the plate and not in the weld. If the yield point of the weld metal is higher than that of the plate, it is possible to bend the

plate through a large angle across the weld, even if the weld has low ductility.

The most desirable combination of characteristics would appear to be obtained when the yield point and ultimate tensile strength of the weld metal are slightly higher than those of the parent metal. If the weld metal is more than 2 or 3 tons per in.^{2} stronger than the parent metal, the uneven stretching of the two metals causes concentrations of stress of unknown magnitude at the junction where the metal is least able to resist the deformations.

Choice of Welding Material. The choice of welding material profoundly affects the design of work, and a comparison of the details of structures built with bare wire and with covered electrodes indicates substantial differences in the method of design for the different classes of material. The non-ductile nature of the bare wire weld precludes the use of the butt weld in highly stressed members, whereas with high-grade electrodes, with which the physical properties of the weld metal approximate to those of the parent plate, the use of butt welds is normal practice.

CHAPTER II

WELD FORMS

Weld Forms and Preparation for Welding. There are two main types of joint—

Butt Welds—joining plates or sections in one plane.

Fillet Welds—joining lapped plates or sections.

Both types of weld may occur in a number of forms. The various forms of both types, and the symbols by which they are indicated on drawings, are set out very completely in *British Standard No.* 499 *Nomenclature Definitions and Symbols for Welding and Cutting*. For fillet welds as a rule no special preparation of plates is required. For butt welds the type of preparation varies according to the thickness of the material and the welding technique adopted.

According to the requirements of the relevant British Standard No. 538—1940, *Metal Arc Welding in Mild Steel as applied to General Building Construction*, the form and "dimensions" of the weld surfaces shall be such as will provide access for the electrode to the surfaces to be welded, and enable the welder to see clearly the work in progress.

For plates under $\frac{3}{16}$ in. thickness no preparation is required. For plates over $\frac{3}{16}$ in. the edges are prepared to form single-V or double-V, single-U or double-U, single-J or double-J, or single-bevel or double-bevel joints, but when a J- or bevel-weld, either of the single or double type, is used, the maximum permissible stresses are reduced to three-fourths of the stresses allowed on V- and U-butt welds.

The plates to be butt-welded must be separated by a gap of at least $\frac{1}{16}$ in. for plates up to $\frac{3}{8}$ in. if the plates are machined to a sharp edge, and $\frac{1}{8}$ in. if the plates are machined to leave a root face, and by a minimum gap of $\frac{1}{8}$ in. for plates over $\frac{3}{8}$ in. In the case of double-V and double-bevel welds, no root

face is permitted. The included angle in the case of V butt welds may not be more than 100°, and according to the position of welding not less than—

60° for flat or downhand welding;
70° for vertical and upright horizontal welding;
80° for overhead welding.

In bevel butt welds the angle of bevel may not be less than 45° nor more than 50°. In the U-butt weld, the radius at the bottom of the U may not be less than $\frac{1}{8}$ in. and the angle of bevel may not be less than 10°. In the J-butt weld, the radius at the bottom of the J may not be less than $\frac{3}{16}$ in., and the angle of bevel may not be less than 20° nor more than 30°.

When plates of different thicknesses are butt welded and the surfaces are $\frac{1}{4}$ in. or more out of line, the thicker plate must be bevelled so that the slope of the surface from one part to the other must not be more than one in five; or, alternatively, the weld metal may be built up at the junction with the thicker part to a thickness at least 25 per cent greater than the thickness of the thinner part.

Single-V, U, J, or bevel welds must, wherever practicable, be finished by depositing a run of weld metal on the back of the joint. Where this is not done, the allowable stress in the weld is reduced to not more than one-half the corresponding stress on a weld with the run on the back. This restriction is necessary because when fusion is not effected right through the plate a sharp notch is left, and under load a high concentration of stress exists at the bottom of the notch (Fig. 3A). It is laid down, however, that, where it is impossible to weld at the back, if the vee is backed by another plate the weld may be considered as developing its full strength if the plates are kept sufficiently far apart to enable fusion to be obtained into the backing plate as well as the two sides of the vee (Fig. 3B).

Butt welds must be reinforced so that the thickness at the centre of the weld is at least 10 per cent more than the thickness of the plates joined. Where a flush surface is required the

butt weld is first reinforced and then dressed flush. When this is done, the allowable working stress in the weld need not be reduced.

These clauses may be taken as representing good practice at the time when the British Standard was prepared. Recent developments in the manufacture and method of using large gauge electrodes have made it possible to depart from these standard preparations with advantage where the conditions are suitable. If large electrodes and a high welding current are used, it is possible to make an effective butt weld in $\frac{3}{8}$ in. or even $\frac{1}{2}$ in. plate without chamfering the plates, for under these circumstances the edges of the plate may be melted to a considerable depth. The U preparation is more economical than

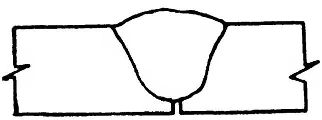


FIG. 3A. NOTCH EFFECT AT BACK OF WELD WHEN FUSION IS NOT COMPLETE

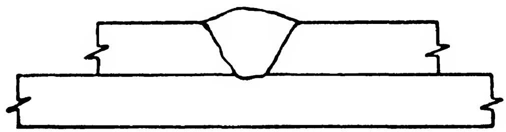


FIG. 3B. PENETRATION EFFECTED IN BACKING PLATE BY LEAVING SUFFICIENTLY WIDE GAP BETWEEN THE PLATES TO BE BUTT WELDED

the V for thick plates, and can be used to advantage if the tools for making the U preparation are available. The U preparation is widely used for the heavy plates of boiler drums and oil-cracking stills. It is frequently useful to chamfer to a sharp edge instead of leaving a square root face. The use of a square root face was due to the fear that the arc playing on the thin edge would overheat it. It would now appear that this danger was exaggerated and that the sharp edge preparation makes for more complete fusion. During the making of a butt weld the two plates tend to come together, and unless they are kept apart there is a tendency for the two edges to be brought hard together. When normal welding currents are used, the depth of penetration below the surface of the plate is only of the order of $\frac{1}{64}$ in. to $\frac{1}{32}$ in., and if two plates are forced close together a root face of any greater depth would remain unwelded. On the other hand, some engineers prefer to use

no gap at all between the plates and to obtain the necessary penetration by the deep penetrative power of the electrode when used with sufficiently high current. The reason for reinforcing the weld is partly to ensure that the depth shall be at least equal to the thickness of the plate, and partly for the annealing effect of the final run on the metal underneath.

Process of Failure of Weld Forms. Acquaintance with the

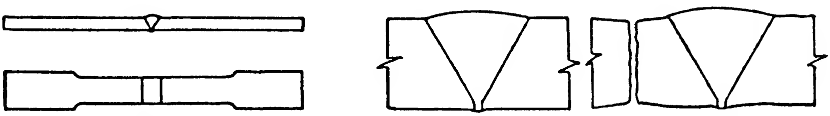


FIG. 3C. FAILURE OF BUTT WELD TEST PIECE WITH REINFORCEMENT LEFT ON

various weld forms and their behaviour under load and at ultimate failure leads to an appreciation of some of the essentials of good design.

BUTT WELDS. Considering the case of a butt joint subjected to a tensile load, when the butt weld is reinforced on both sides of the plate, the section through the weld is increased to such an extent that it is unlikely for failure to occur in the weld, and the fracture normally occurs some distance away

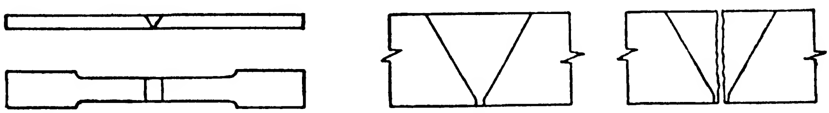


FIG. 3D. FAILURE OF BUTT WELD TEST PIECE WITH WELD GROUND FLUSH IF TENSILE STRENGTH OF WELD METAL IS LESS THAN THAT OF PLATE

(Fig. 3c). The reinforcement acts as a supporting rib which inhibits deformation or "necking" in the immediate vicinity of the weld. If the weld is ground flush with the surface of the plate, the position of the fracture is determined by the relative strengths of the plate and the weld metal. If the tensile strength or the yield point is lower for the weld metal than for the plate, failure takes place through the centre of

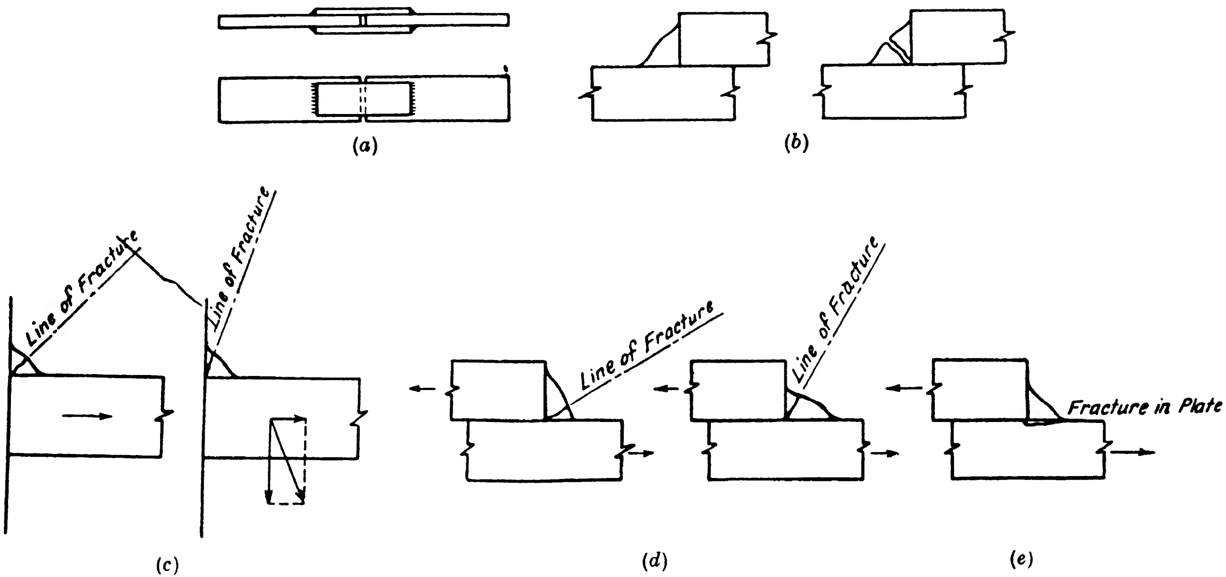


FIG. 4. POSITION OF FRACTURE IN END FILLET WELDS

the weld (Fig. 3D), but if the tensile strength and yield point of the weld are higher, failure takes place in the plate away from the weld. Failure in the weld junction is quite unusual.

Under conditions of free-bending across a reinforced butt weld, the stiffening effect of the reinforcement inhibits failure at the joint. There is some tendency for a crack to start at the weld junction owing to the concentration of stress due to change of section at the edge of the reinforcement, particularly if there is any undercutting. With a flush butt weld from which the reinforcement has been removed, failure under free-bending

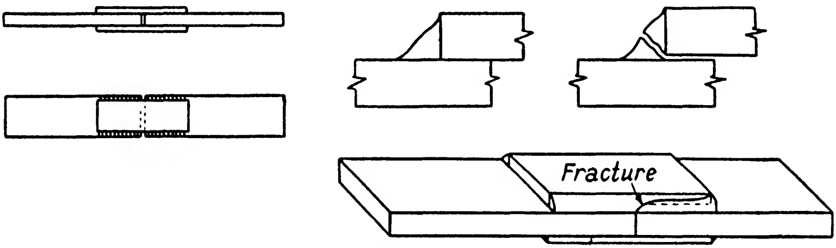


FIG. 5. POSITION OF FRACTURE IN SIDE FILLET WELDS

usually occurs at the middle of the weld, unless there is a wide difference in tensile strength or yield point between the two metals, in which case failure may occur at the weld junction. With a weld metal of high quality, there is no difficulty in bending a butt-welded specimen in mild steel plate across the weld round a former having a diameter of twice or three times the thickness of the plate.

END FILLETS. With an end fillet of normal profile, the plane of fracture under a tensile load is along the diagonal from the root of the fillet (Fig. 4 (*b*)). When the fillet is subjected to a shear stress in addition to the tensile stress the position of the line of fracture departs from the diagonal according to the relative magnitude of the two stresses (Fig. 4 (*c*)).

If the legs of the fillet are unequal, fracture usually occurs near the shorter leg as in Fig. 4 (*d*). If the tensile strength of the weld metal is considerably greater than that of the

plate, the fillet may remain intact and be pulled right out of the plate as in Fig. 4 (*e*). With all end fillets failure occurs abruptly, after a small amount of deformation.

Side Fillets. With side fillets of normal profile subjected to a shear stress along the weld, failure occurs down the throat of the weld. The break commences at the toe of the fillet at one or both ends of the weld, and, as it progresses, the plane of fracture rotates to the diagonal, as indicated in Fig. 5. The failure is gradual, and considerable deformation of the fillet and usually also of the plates takes place before final fracture occurs. The amount of deformation which takes place before fracture commences is, however, the same as for end fillets.

CHAPTER III

STRESSES IN WELDS AND WELDED JOINTS

Butt Welds. If the physical properties of the weld metal in a butt weld approximate to those of the parent metal, it is apparent that the stress in the weld metal can be calculated simply, and the stress in the throat (Fig. 6) is given by—

$$s = P/lt$$

where s = stress per in.^{2} of throat area;
P = the applied load;
l = the length of the weld;
t = depth of throat (i.e. plate thickness).

Any reinforcement on the weld is ignored in calculating the stress. As has been pointed out, the final run of metal on any weld is of coarser crystal structure than the runs underneath, which are partly annealed by the runs above. The provision of reinforcement ensures that the depth through the weld is at least equal to the thickness of the plate and, in the case of thick plates, that the metal within the thickness of the plate is in the annealed condition. Subsequent removal of the reinforcement is not considered as reducing the strength of the joint.

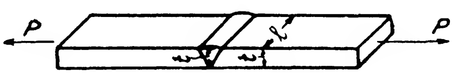


FIG. 6. BUTT WELD UNDER TENSION LOAD

Since the butt weld involves no abrupt change in section at the joint, it is the most suitable form of weld for transmitting alternating stresses. In a rotating bar test in which the butt-welded piece is machined to a smooth surface, the strength of the weld metal from a good electrode is little inferior to mild steel. If a test piece is made with the reinforcement left

on, the concentration of stress due to the change of section and surface defects at the edge of the reinforcement leads to failure at a much lower figure.

End Fillet Welds. For static loading, the stress in end fillet welds is determined by dividing the load transmitted by the area of the throat of the fillet. The throat of the fillet is usually considered as being the depth of the largest right-angle triangle which can be enclosed in the weld, measured along the

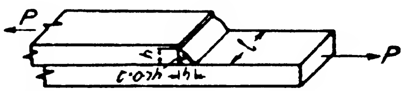


FIG. 7. END FILLET WELD UNDER TENSION LOAD

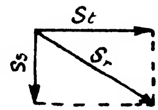


FIG. 8. VECTORIAL ADDITION OF STRESSES IN END FILLET WELDS

bisector of the right angle. The depth of the throat is therefore $t = 0{\cdot}7h$ where h is the length of the leg of the fillet (Fig. 7). The stress on the throat is given by the equation—

$$s = P/lt = P/0{\cdot}7lh.$$

where s = tensile stress in the throat;
P = the load transmitted by the weld;
l = the length of the weld;
t = the throat thickness of the weld;
h = the length of the leg of the fillet.

If the fillet is acted upon by a combination of forces, as for example a tensile load and a shear load at right angles to it, the resultant stress on the throat is obtained by adding the separate stresses vectorially (Fig. 8).

$$s_r = \sqrt{(s_t^2 + s_s^2)}$$

where s_t = the stress due to the tensile load;
s_s = the stress due to the shear load;
s_r = the resultant stress.

This method of calculation is admittedly an approximation, but is simple and safe. More precise methods of calculation

have been suggested, but as the results have not been shown to conform any more closely to the results of practical tests, there would appear to be no valid reasons for using them.

Photoelastic studies of end fillet welds under static stress indicate that at small loads considerable concentrations of stress exist at the heel and toe of the shear face of the fillet. As the load is increased, the yield point is reached at the points of stress concentration, and the load is transferred to other parts of the weld so that at ultimate load the stress is fairly uniform over the throat section. Under alternating stress no such redistribution of stress is possible, and for alternating stresses this method of calculation is not applicable.

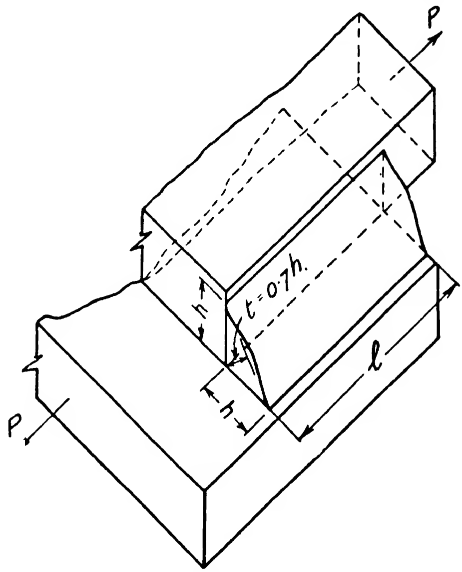


FIG. 9. SIDE FILLET WELD UNDER TENSION LOAD

Experiments on fillet welds under alternating stress indicate that the fatigue strength depends very greatly on the shape of the fillet, and concave fillets give much better value than convex fillets, though the convex fillets are substantially stronger under static loads.

Side Fillet Welds. The stress on the throat of a side fillet weld (Fig. 9) is determined by the equation—

$$s = P/lt = P/0{\cdot}7lh.$$

where s = shear stress in the throat of the weld. As in the case of end fillets, this method of calculation assumes that, before the ultimate load is reached, sufficient plastic yield will have

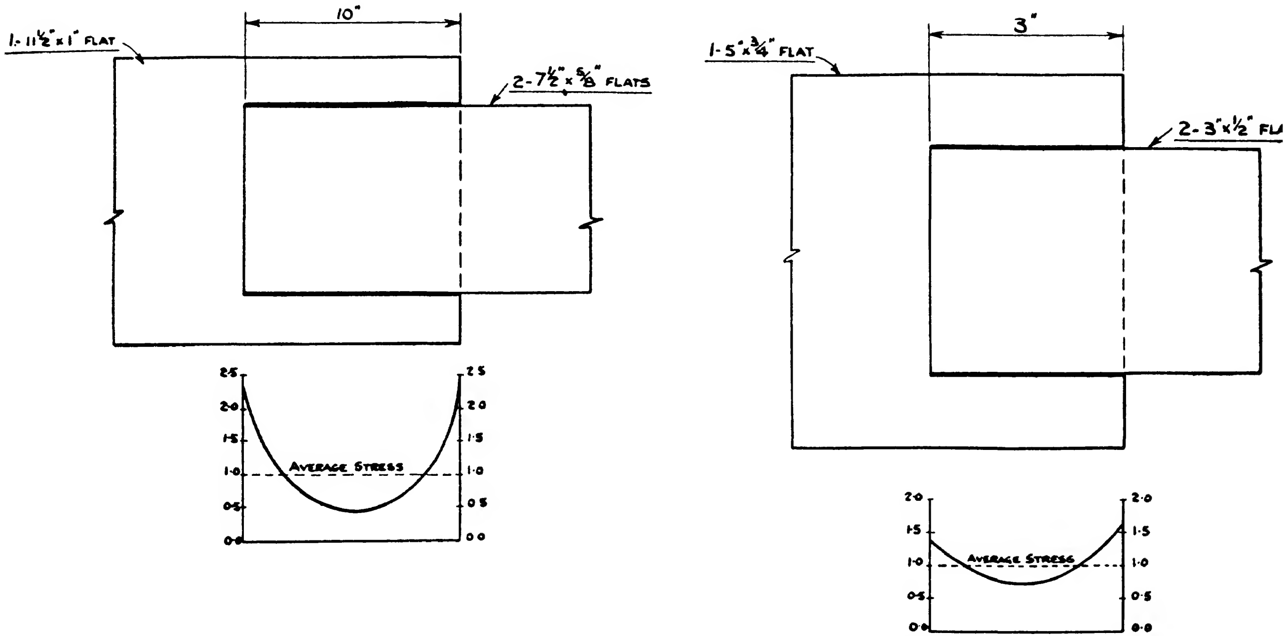


Fig. 10. Distribution of Stress along Side Fillet Welds under Load

taken place in the weld metal or the adjacent plate to allow the stress to become uniform over the section. It is established by experiment, as well as by theoretical calculation, that within the elastic limit the stress is not evenly distributed throughout the length of the weld, the ends being more highly stressed than the middle. The distribution of stress along a side fillet weld joining two flats which are subjected to a tension load is shown in Fig. 10. The theoretical stress at the ends is about five times that at the middle of the weld, the maximum stress being two and a half times the average. If the material of the weld behaved elastically throughout its whole range of strength, this condition would be highly undesirable, and would seriously limit the usefulness of this type of weld, for the fillet would fail progressively from the ends at a comparatively low stress. That this does not occur is established quite simply by the fact that side fillet welds fail at a stress of about 20 tons per in.^{2} on the throat area, irrespective of the size or length of the welds, that is to say, at the normal shear strength of the material.

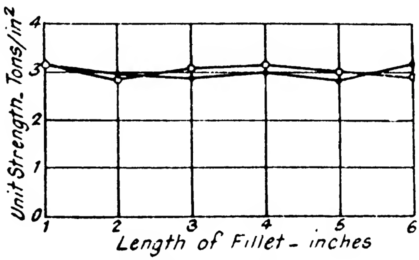


FIG. 11. ULTIMATE STRENGTH OF SIDE FILLET WELDS OF VARIOUS LENGTHS

In a series of tests on side fillet welds varying from 1 in. to 6 in. it was shown that, provided the plates of the test piece are not stressed beyond their elastic limit, the strength of the side fillet is independent of the length. The results of the tests are shown in graphical form in Fig. 11.* If the metal behaved elastically throughout, the strength of the welds would decrease as the length increased. Examination of a weld which has fractured in longitudinal shear indicates that the weld metal

* R. R. Blackwood: "Strength of Fillet Welds in Structural Mild Steel, I," *Commonwealth Engineer*, 1930–31. Vol. 18. No. 2, pp. 50–55; No. 3, pp. 89–97.

actually deforms before failure by an amount very much greater than that required to redistribute the stress evenly along the fillet. Fig. 12 is a diagrammatic representation of a side weld test piece after test, in which the plates have drawn apart about $\frac{1}{8}$ in. without fracture occurring. The amount of movement necessary to redistribute the stress along a fillet 6 in. long is of the order 0·01 in.

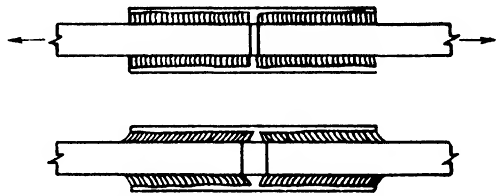


Fig. 12. Sketch Indicating Ability of Side Fillet Welds to Undergo Deformation

That a small deformation of limited amount may take place in a steel member of whatever shape without any ill effect may readily be seen by considering the stress-strain diagram for a tensile test specimen. When a bar is subjected to a stress greater than the elastic limit a permanent extension takes place, and if the load is applied repeatedly so that a number of extensions take place, the limit of extension of the bar is reached and fracture occurs. Fig. 13A shows the manner in which the total extension of the bar is exhausted by a number of successive loadings. It is on the assumption of such behaviour that the method of determining working stresses as a fraction of the elastic limit is based. If, however, the method of loading is such that the bar can never be stretched beyond a definite limited amount, the conditions are quite different. It is necessary to consider two cases. If after the initial loading and stretching by a limited amount the bar is subsequently stressed by a smaller amount, it will behave

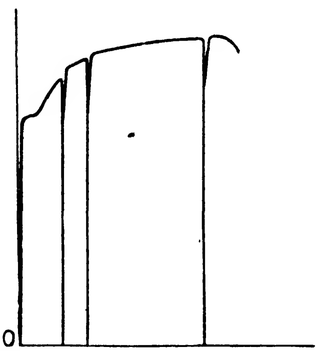


Fig. 13A. Load-strain Diagram for Repeated Applications of Load beyond Yield Point when Extension is Not Limited

elastically throughout because the elastic limit of the steel is raised by the initial stretching (Fig. 13B), but if the bar under consideration is a member of a rigid frame, the behaviour is indicated in Fig. 13C. Under the first loading beyond the elastic limit, the bar stretches permanently by a limited amount, and any load beyond that which the bar is capable of carrying elastically is transferred to the other members of the

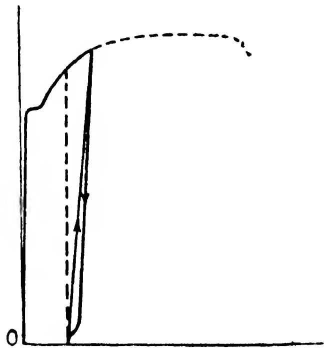


FIG. 13B. LOAD-STRAIN DIAGRAM FOR REPEATED APPLICATIONS OF LOAD WHEN THE EXTENSION IS LIMITED TO A DEFINITE AMOUNT

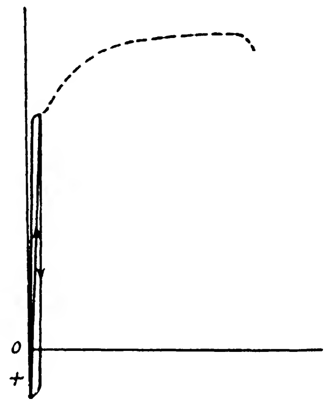


FIG. 13C. LOAD-STRAIN DIAGRAM FOR REPEATED APPLICATIONS OF LOAD WHEN MEMBER IS PART OF A RIGID FRAMEWORK

frame. On releasing the external load, the elongated bar is subjected to a load from the other members of the frame in the opposite sense to the load it carried originally. When there is no external load in the bar it is then in a state of compression, and on subsequent reloading with tension load it behaves elastically over a greater range of stress. In a similar way, when the stress at the ends of a side fillet weld passes the elastic limit, the metal yields by a small limited amount and transfers part of the load to the middle of the weld.

Since the amount which the weld metal is called upon to stretch in distributing the load along the weld is well within

the range of extension of the metal, the existence of stress concentration at the end of the weld at the first loading is not a matter of great importance in so far as static loading is concerned. On the other hand, it is clear that for complete reversals of stress the existence of an initial compression at the ends of the weld after a first tension loading will reduce considerably the elastic range of the specimen for compression loading. During the application of the initial load to the

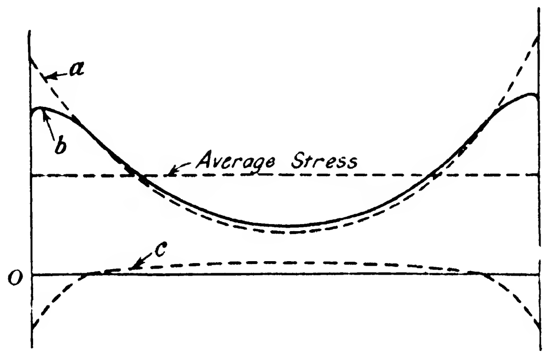


FIG. 14. DISTRIBUTION OF STRESS IN SIDE FILLET WELD AFTER INITIAL LOADING AND AFTER RELEASE OF LOAD

specimen, the distribution of stress along the fillet is as indicated by the dotted line *a* in Fig. 14, as long as the stress remains within the elastic limit. When the stress at the ends of the fillet reaches the elastic limit, no further increase is possible until the metal has taken a large permanent set, so the load is distributed along the fillet, as indicated by the full line *b*. Permanent deformation at the ends of the fillet results, and, when the load is removed, an initial compression remains in the ends of the weld balanced by a tension in the middle as indicated by the dotted line *c*. If a specimen so strained is loaded in the opposite sense, the concentration of stress at the ends of the fillet will be augmented by the initial compression and the elastic limit of the material will be reached at a comparatively small load. Uneven distribution of stress within the elastic

limit of the material must not be ignored when dealing with alternating stresses, for the extremities of a weld may conceivably be repeatedly stressed above the yield point of the metal without the average stress approaching the yield point, and under these circumstances progressive failure of the metal may take place. For alternating stresses involving complete reversal of stress, a fillet weld does not constitute a desirable

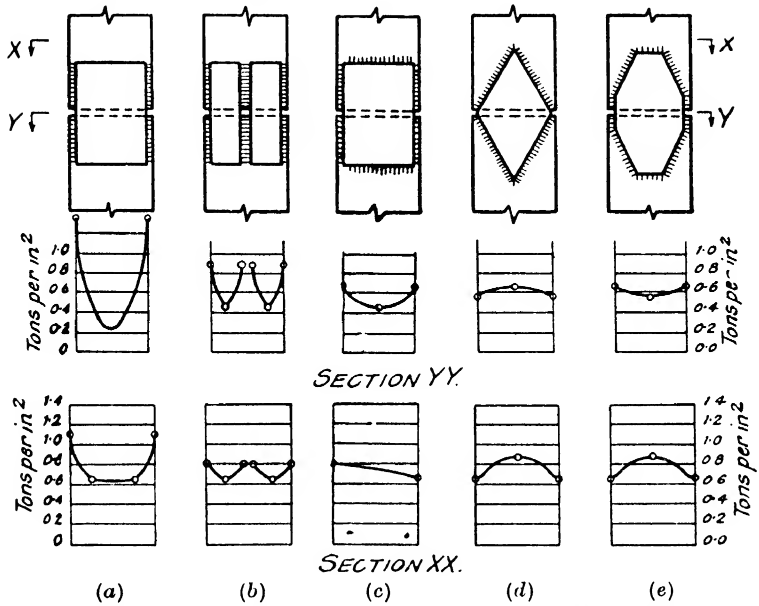


Fig. 15. Distribution of Stress in Welded Butt-strap Joints

form of joint, and should only be used after very careful consideration and for low working stress.

It is necessary to give most careful attention to the possibility of stress concentrations in any work subject to alternating stresses, but alternating stresses very seldom occur in building structures.

Distribution of Stress in Welded Joints. In addition to the stress concentrations in and immediately adjacent to the welds, it is frequently necessary to take into consideration how the

disposition of the welds may affect the stress distribution in the members joined. The results of a series of experiments on the distribution of stress in a number of types of welded butt-strap joints are indicated in Fig. 15.* It is seen that if the welds are parallel with the lines of stress and are placed at the edges of the plates, there is a serious concentration of stress at the edges of the plates. It is found that when the splice plates are welded on the sides only, and particularly when the plates are wide, failure occurs by tearing from the edges of the plate at quite a low average stress (Fig. 15 (*a*)). By dividing the

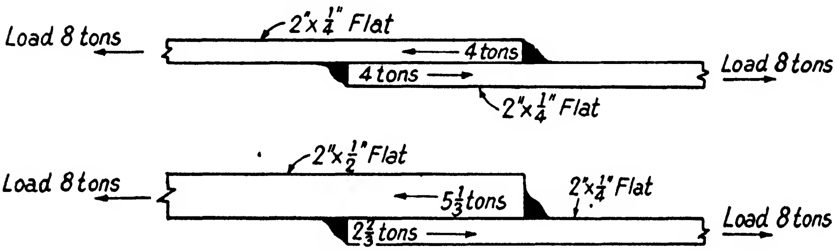


FIG. 16. DISTRIBUTION OF STRESS BETWEEN END WELDS IN LAP WELDED JOINT BETWEEN PLATES OF DIFFERENT THICKNESS

cover plates into two (Fig. 15 (*b*)), and thus putting additional welds down the middle of the main plates, the distribution across the main plate is improved. Much more uniform distribution is obtained when end welds are used in conjunction with the side welds (Fig. 15 (*c*)). The use of diamond-shaped cover straps results in a very uniform distribution of stress in the plates, as might be expected (Fig. 15 (*d*)). The diamond-shaped cover strap is, however, not so satisfactory in so far as the weld itself is concerned, as a heavy concentration of stress exists in the weld at the point of the diamond. While the results are those obtained with a direct tension load only, and the distribution of stress under other methods of loading may be different, the tests indicate clearly the necessity for analysing

* "Distribution of Stresses in Welded Butt-strap Joints"; Hollister and Gelman, *Jnl. of American Welding Society*, 1932. Vol. II No. 10, pp. 24–31.

the joints carefully in order to avoid stress concentrations of serious magnitude.

As a rule it may be assumed that under static loading the weld metal will adjust itself to any internal inequalities of stress, but it is always necessary to give careful attention to the elastic behaviour of the piece as a whole, for the stresses on

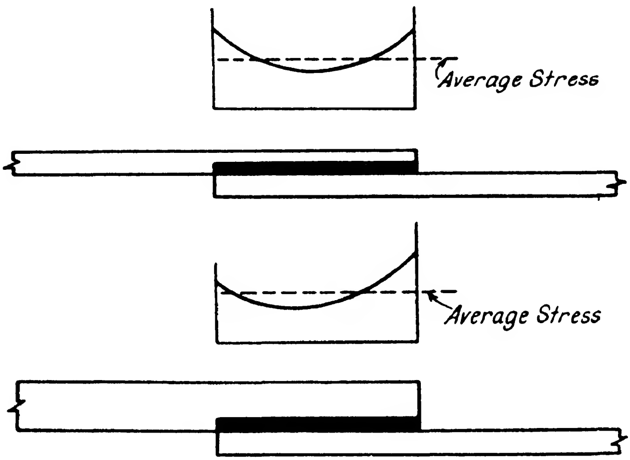


FIG. 17. DISTRIBUTION OF STRESS ALONG SIDE FILLET WELDS JOINING PLATES OF DIFFERENT THICKNESS

the different welds in a joint may depend very largely on the shape and relative thickness of the pieces joined.

Consider as an example two flats joined with end fillet welds (Fig. 16). When the plates are of equal thickness the load will be evenly distributed between the two plates, and between the two welds, but when the plates are of different thicknesses the load carried by each will be proportional to its thickness, and the welds at the ends of the plates will carry the same proportions of the load. In the example cited, the weld at the end of the heavier plate would carry twice as much load as the other weld.

Similarly with side welds the width and thickness of the two

plates alter the distribution of stress along the welds as indicated in Fig. 17.

While there is no standard method of dealing with these problems, it is possible to allow for the inequality of stress distribution with a reasonable degree of accuracy if the general behaviour is understood.

CHAPTER IV

DESIGN DATA

FOR purposes of design, tables of strength of welds may be prepared. According to British Standard Specification 639, *Covered Electrodes for Metal Arc Welding Wrought Iron and Mild Steel (for hand operation)*, the physical properties required of the weld metal for Class A electrodes, which are specified for use in the welding of structural steel work, as determined from a specimen built entirely of deposited metal, must not be less than the following—

Ultimate tensile strength . .	28 tons per in.^{2}
Elongation on 3·54 diameter . .	Not less than 20 per cent
Reduction of area	Not less than 35 per cent
Izod impact value 10 mm. specimen.	Not less than 30 ft. lb.

The maximum allowable working stresses as set out in the *British Standard Specification for Metal Arc Welding in Mild Steel as applied to General Building Construction (revised April*, 1940), are as under—

Classification of Stress in Welded Connections	Maximum Permissible Stress (Tons per In.^{2})
For tensile and compressive stress in butt welds	8
For shear stress in butt welds	5
Stress in end fillet welds in lap joints	7
Stress in all other fillet welds	5

In the Regulations made under Section 9 (2) of the London Building (Amendment) Act, 1935, relating to applications for modification or waiver of the building by-laws, so as to permit

the use of electric (metal) arc welding instead of riveting, bolting or lapping, it is laid down that "the Council will determine in each case the maximum permissible stresses, the detail arrangement of connections and such other restrictions as the Council may deem proper for the use of such welding in the manner proposed."

The following table may, however, be taken as a general indication of the probable maximum stresses which will be permitted by the Council—

Classification of Stress in Welded Connections	Maximum Permissible Stress (Tons per In.^{2})
Tension and compression in butt welds	8
Shearing in butt welds in webs of plate girders and joists .	6
Shearing in butt welds other than webs of plate girders and joists	5
Stress in end fillet welds	6
Stress in side fillet welds, diagonal fillet welds, and tee fillet welds	5

In Tables III and IV the ultimate strengths in tension, compression, and shear, for butt welds of different sizes, assuming a maximum stress of 8 tons per in.^{2} in tension and compression and 6 tons per in.^{2} in shear, are given.

The permissible stresses tabulated above are applicable to static loading. No specification covering alternating repeated or pulsating loading has yet been prepared, nor is any projected at present. In the tentative specification of the German railways for welded plate girder railway bridges, factors are given which vary the permissible stresses for the different types of loading and for various parts of structures according to the nature of the stress they carry. It is doubtful whether the available knowledge warrants such precision of treatment, but there is much to be learned from a study of this specification and the tests on which it was based.*

* *Bauverwaltung*, 1935, 55(50), 1008–23.

TABLE III

STRENGTH OF BUTT WELDS BASED ON 8 TONS PER IN.^{2} FOR TENSION OR COMPRESSION, AND 5 TONS PER IN.^{2} FOR SHEAR

Plate Thickness (in.)	Strength per Linear Inch of Weld		
	Ultimate	Working Stress	
		Tension and Compression	Shear
	Tons per Linear Inch		
$\frac{1}{8}$	3·5	1·0	0·62
$\frac{3}{16}$	5·25	1·5	0·93
$\frac{1}{4}$	7·0	2·0	1·2
$\frac{3}{8}$	10·5	3·0	1·8
$\frac{1}{2}$	14·0	4·0	2·5
$\frac{5}{8}$	17·5	5·0	3·1
$\frac{3}{4}$	21·0	6·0	3·7
$\frac{7}{8}$	24·5	7·0	4·3
1	28·0	8·0	5·0

Probable Variation in Strength of Welds. In the Report of the Structural Steel Welding Committee of the American Bureau of Welding, a committee formed in 1926 for the purpose of "obtaining reliable information on which to base safe unit working stresses in the designing of welded structures," it is stated that "in commercial practice a welded joint may be expected to have a strength within 12 per cent of a general average for that type of joint." The tests on which this statement is based embraced some 1 400 tests on 55 elemental types, welded by 61 welders belonging to 39 fabricating shops. These tests were made with bare electrodes with which the probable variation from the average strength is greater than with covered electrodes.

For information on the probable variation in the strength of welds, reference may be made to the statistical investigation of the strength and reliability of welded joints carried out by the Welding Panel of the Steel Structures Research Committee.*

* Report of the Welding Panel of the Steel Structures Research Committee, 1938.

Standard deviation = (number of observations)
× sum of (observed value — mean value).

Mean deviation is the sum of deviations from the mean, irrespective of sign, divided by the number of observations.

For Grade A electrodes—those giving a minimum tensile strength of 28 tons per in.^{2} and an elongation of 20 per cent (measured on a gauge length equal to 3·54 times the diameter of the test piece) in the all weld metal test piece, and a minimum of 30 ft.-lb. in the all weld metal notched bar impact test—the following results were obtained for fillet welds.

TABLE IV

Strength of Fillet Welds Based on 7 Tons per In.^{2} for end Fillets and 5 Tons per In.^{2} for Side Fillets

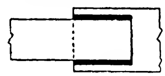
Fillet Leg Size (in.)	Throat Thickness (in.)	Strength of Weld in Tons per Linear Inch			
		End Welds 		Side Welds 	
		Ultimate Strength	Working Stress	Ultimate Strength	Working Stress
$\frac{1}{8}$	0·088	2·4	0·6	1·7	0·4
$\frac{3}{16}$	0·133	3·6	0·9	2·7	0·6
$\frac{1}{4}$	0·176	4·8	1·2	3·5	0·9
$\frac{3}{8}$	0·265	7·2	1·8	5·3	1·3
$\frac{1}{2}$	0·354	10·0	2·5	7·0	1·8
$\frac{5}{8}$	0·441	12·8	3·2	8·8	2·2
$\frac{3}{4}$	0·53	14·8	3·7	10·8	2·7
$\frac{7}{8}$	0·618	17·2	4·3	12·4	3·1
1	0·707	19·6	4·9	14·0	3·5

TABLE V

GRADE A ELECTRODES—VALUES OF ULTIMATE STRESS
(Tons per In.^{2})

Type of Weld	Welding Position	No. of Tests	Lowest Value	Highest Value	Mean Value	Standard Deviation		Mean Deviation	
						Actual	Percentage of Mean	Actual	Percentage of Mean
Transverse fillets	Horizontal	136	14·2	51·0	34·1	6·2	18·3	4·7	13·8
	Vertical .	90	17·1	45·2	30·3	6·3	20·7	5·1	16·9
	Overhead.	80	17·8	64·9	35·2	6·8	19·3	4·9	13·9
	Total .	306	14·2	64·9	33·2	6·7	20·1	5·1	15·2
Longitudinal fillets	Horizontal	138	16·2	31·1	22·6	2·5	11·1	2·0	9·0
	Vertical .	90	14·0	32·9	22·9	3·6	15·9	2·8	12·2
	Overhead.	81	16·2	34·6	24·7	3·6	14·8	3·0	12·2
	Total .	309	14·0	34·6	23·2	3·3	14·2	2·5	10·9

For butt welds, the following results were obtained—

TABLE VI

REDUCED SECTION TENSILE TESTS—ULTIMATE STRESS VALUES
(Tons per In.^{2})

Welding Position	Electrodes	No. of Tests	Lowest Value	Highest Value	Mean Value	Standard Deviation		Mean Deviation	
						Actual	Percentage of Mean	Actual	Percentage of Mean
Horizontal welding	Grade A electrodes	45	27·3	38·8	33·4	2·7	8·2	2·1	6·4
Vertical welding	Grade A electrodes	30	19·7	37·0	30·0	4·1	13·7	3·4	11·5

These limits of variation are well within the limits required for ordinary engineering construction.

Strength of Single-run Fillets. Special reference should be made to the strength of single-run fillets, which constitute the bulk of the welding in constructional work. The strength of the single-run fillet is not necessarily proportional to the

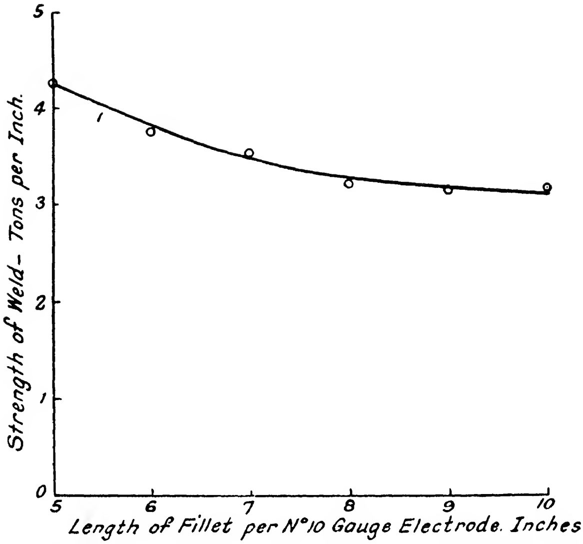


FIG. 18. STRENGTH OF FILLET WELDS MADE WITH VARIOUS LENGTHS OF RUN PER ELECTRODE

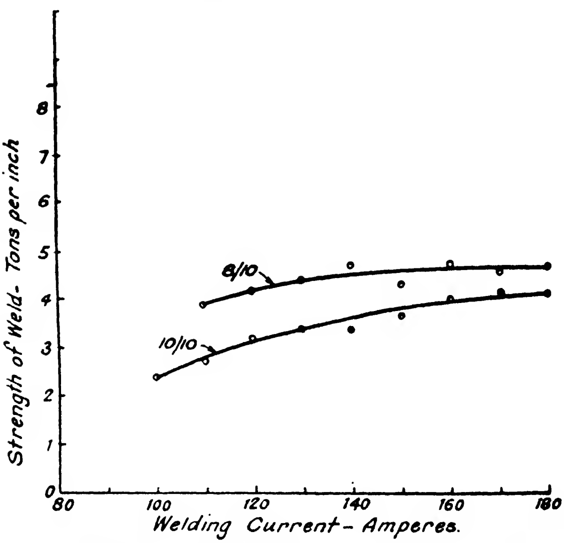


FIG. 19. VARIATION OF STRENGTH OF WELDS WITH DIFFERENT WELDING CURRENT

measured linear dimensions. In Fig. 18 the results of a series of tests on end fillet welds from 10/5 to 10/10 (approximately $\frac{1}{4}$ in. to $\frac{3}{16}$ in.), made with a covered mild steel electrode, are given in graphical form.* The strength of the smaller fillets is proportionately greater than that of the larger fillets. On examining cut specimens of the welds, it will be seen that the greater strength of the smaller fillets is due to the greater penetration which is achieved with the small runs. In making the smaller fillets, the electrode tip is moved forward without oscillation, a close arc is held, and fusion is effected right into the corner. In making the larger runs, the electrode tip is weaved to spread the metal and, since the arc does not play on the root, penetration is not achieved. If fusion can be effected beyond the corner, additional throat thickness and, consequently, greater strength are obtained. The strength of single-run fillets is also dependent on the welding current used. Within practical limits the strength of all sizes of single-run fillets increases with increase in current. As the current is increased, the depth of penetration is increased and with it the throat thickness on which the strength of the weld depends. The results of a series of tests on the relation between the welding current and the strength of single-run end fillet welds are given graphically in Fig. 19.†

* See R. R. Blackwood: "Strength of Electric Arc Welds in Structural Mild Steel, I," *Commonwealth Engineer*, 1930–31, Vol. 18, No 2, pp. 50–55; No. 3, pp. 89–97.

† See R. R. Blackwood: "Strength of Electric Arc Welds in Structural Mild Steel, II," *Journal of the Australian Welding Institute*, June and July, 1932.

CHAPTER V

DESIGN OF TYPICAL JOINTS

IN discussing the design of welded joints three separate conditions must be considered: (1) Joints carrying direct stress—tension, compression, or shear; (2) joints carrying direct stress together with bending stress due to eccentricity; (3) rigid joints transmitting bending moments with or without direct stress.

Joints Carrying Direct Stress. (*a*) SECTION SYMMETRICAL ABOUT AXIS. For connections of the type shown in Fig. 20, in which the member is symmetrical about an axis parallel to the line of stress, and the welds can also be placed symmetrically about the axis, the required length of weld is given by $l = P/s$, where

$l =$ length of weld;
$P =$ load on the joint;
$s =$ working stress per linear inch of weld.

EXAMPLE 1. Two tie-bars are to be attached to another bar by end welds (Fig. 21). The length of the end welds is determined by the width of the bars and it is required to find the size of fillet necessary to carry the given load.

In Fig. 21, load $P = 8$ tons. Total length of weld $l = 8$ in. Therefore the load per inch of fillet $= 8/8 = 1$ ton. From Table IV, the permissible working stress on $\frac{1}{4}$ in. end fillet $= 1{\cdot}2$ tons per linear inch, and this is satisfactory. In practice it is usual, when possible, to carry the welds round the corner and the returned part of the weld is not taken into account.

EXAMPLE 2. Two tie-bars are to be attached to a gusset plate by means of $\frac{1}{4}$ in. side fillet welds (Fig. 22). It is required to find the length of weld necessary to carry the given load.

In Fig. 22, the load in each strap is $P = 9$ tons. From Table

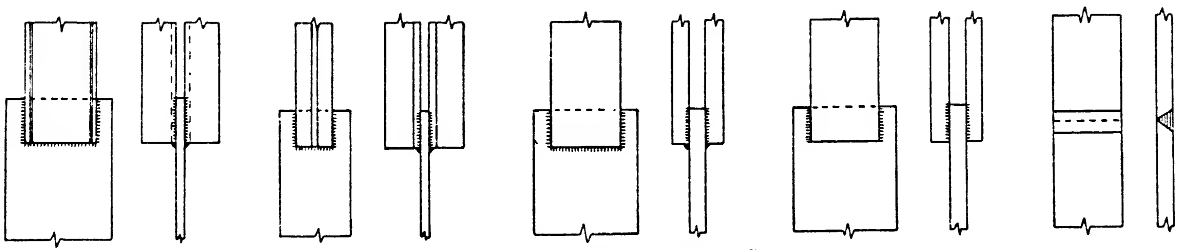


FIG. 20. WELD CARRYING DIRECT STRESS

IV, page 30, the working stress of a $\frac{1}{4}$ in. side fillet $= 0{\cdot}9$ ton per linear inch. Therefore the length of weld required is $9/0{\cdot}9 = 10$ in. According to B.S. 538 a length of weld equal to the nominal size of the fillet at each end of the weld must be disregarded in calculating the length of weld required. If it is not possible for the weld to be returned round the end of the bar it will be necessary for the welds to be made $5\frac{1}{2}$ in. long on each side of the bar. Welds 5 in. long are placed one on each side of the bar.

The length of the side fillets on ties of flat section should be

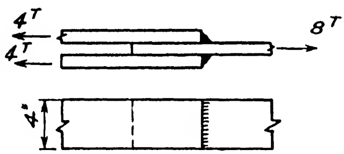


FIG. 21. TIE-BARS ATTACHED BY END FILLET WELDS

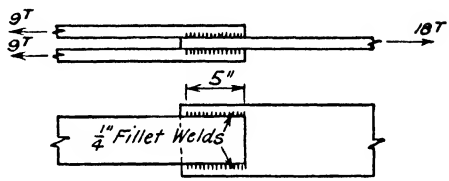


FIG. 22. TIE-BARS ATTACHED BY SIDE FILLET WELDS

not less than the width of the flat in order to limit the concentration of stress at the edges of the flat as described on page 24. The unevenness of the stress distribution referred to is accentuated as the width of the plates is increased. For this reason the distance between lines of side welds is limited to 30 times the thickness of the plates, and if the plates are wider than 30 times their thickness, slot welds must be introduced. Side fillet welds in slots have the same value as ordinary side welds.

EXAMPLE 3. It is required to join two hangers 16 in. $\times \frac{1}{2}$ in. by means of side fillets (Fig. 23). As the width of the plates is $16/0{\cdot}5 = 32$ times the thickness it is necessary to insert a slot. The load in the flat when stressed to 8 tons per in.$^2 = 16 \times 0{\cdot}5 \times 8 = 64$ tons. From Table IV the strength of $\frac{1}{2}$ in. side fillet welds is $1{\cdot}8$ tons per linear inch. The length required for each of the four welds is therefore $64/(1{\cdot}8 \times 4) = 8{\cdot}89$ in.

In order that the effective area of the plate shall not be

reduced, the slot is set back behind the beginning of the welds on the edges of the plate. Area of slot $= 0{\cdot}5$ in.^{2} Welds must extend in front of slot by $0{\cdot}5 \times 8/(1{\cdot}8 \times 2) = 1{\cdot}1$ in.

Care must be taken in using slot or notch welds that the effective area of the plate is not reduced and that the disposition of the slots or notches is not such as to lead to serious concentrations of stress within the plates. As a rule the provision of a slot tends to improve the distribution of stress in the plates

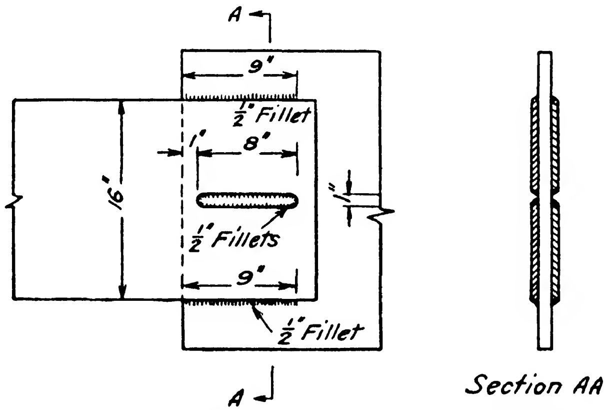


FIG. 23. TENSION BAR ATTACHED BY SIDE WELDS AND SLOT WELDS

as in Fig. 24. On first sight the joint of Fig. 25 (*b*) might appear to be greatly superior in strength to that of Fig. 25 (*a*) but it was found under test that the joint (*b*) failed in the manner indicated in Fig. 25 (*c*) at a much lower load than joint (*a*) owing to the concentrations of stress in the plate in the re-entrant angles of the notches.

EXAMPLE 4. Two channels are to be attached to a gusset plate by means of a combination of side and end fillets (Fig. 26). For practical purposes the end and side welds may be considered to carry the load in proportion to their separate static strengths. The length of the end fillet weld is determined by the depth of the channel and the size is limited to ¼ in.—the thickness of the web of the channel. It is required to find the

length of $\frac{3}{8}$ in. side fillet welds which would be necessary to carry the part of the load not carried by the end welds.

In Fig. 26 the load in each channel is 16 tons. From Table

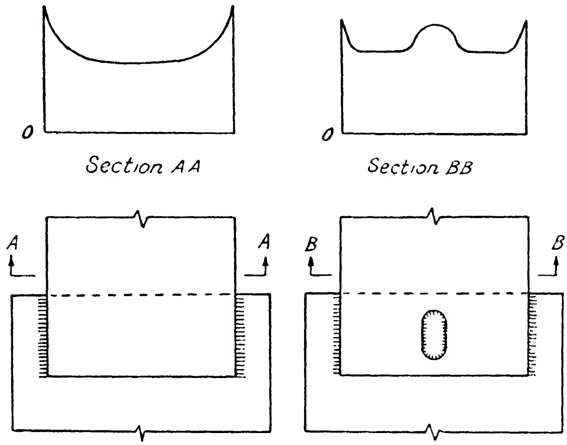


FIG. 24. DISTRIBUTION OF STRESS IN SIDE-WELDED TENSION BAR WITH AND WITHOUT SLOT

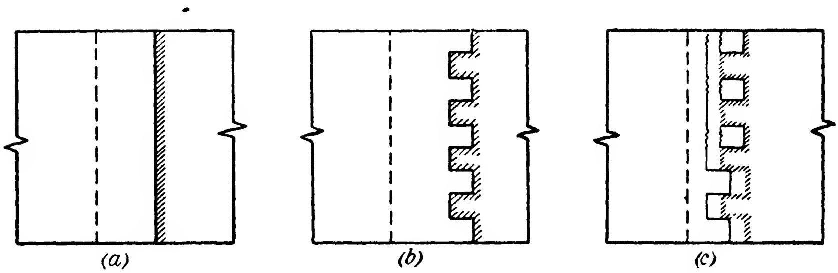


FIG. 25. UNSATISFACTORY METHOD OF INCREASING LENGTH OF FILLET WELDS BY MEANS OF SQUARE SLOTS

IV the $\frac{1}{4}$ in. end fillet weld carries a load of 1·2 tons per linear inch. Therefore the load carried by the end fillet $= 4 \times 1{\cdot}2 = 4{\cdot}8$ tons leaving $16 - 4{\cdot}8 = 11{\cdot}2$ tons to be carried by the two side welds. The strength of the $\frac{3}{8}$ in. side fillet is 1·3 tons per linear inch. Therefore the length of the side weld required

$= 11{\cdot}2/1{\cdot}3 = 8{\cdot}6$ in. The channel is made to overlap the edge of the gusset by $4{\cdot}3$ in. $+ 0{\cdot}375$ in. $= 4{\cdot}67$—say 5 in.

EXAMPLE 5. The strength of oblique fillets is intermediate between the strength of end and side fillets, the strength of the diagonal fillet being a mean between the two values. Diagonal fillets may be considered to act with end or side fillets in proportion to their strengths.

In Fig. 27 two flat bars are joined to a gusset plate by means of a combination of side and diagonal fillets. The load in each bar is 12 tons.

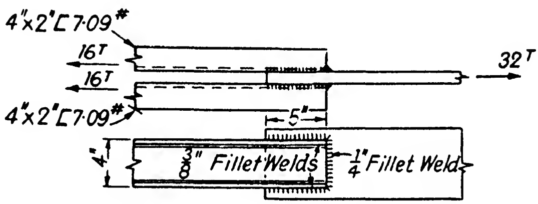


FIG. 26. TENSION BARS ATTACHED BY SIDE AND END FILLETS

Length of diagonal weld $= 2{\cdot}8 \times 2 = 5{\cdot}6$ in. From Table IV the strength of the $\frac{3}{8}$ in. diagonal fillet taken as a mean of the values for the end and side welds $= 1{\cdot}65$ tons per linear inch. Load carried by diagonal weld $= 5{\cdot}6 \times 1{\cdot}65 = 9{\cdot}14$.

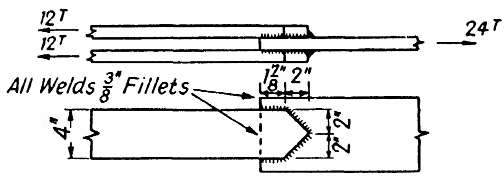


FIG. 27. TENSION BARS ATTACHED BY SIDE AND DIAGONAL FILLETS

Therefore the load in the side welds is $12 - 9{\cdot}14 = 2{\cdot}86$ tons, and the effective length of the side welds required $2{\cdot}86/1{\cdot}3 = 2{\cdot}2$ in., and the length needed will be $1{\cdot}1 + 0{\cdot}375 = 1{\cdot}475$ in.

In the examples given above it is assumed that the members are joined in such a way that there is no significant eccentricity.

If plates are lapped as in Fig. 28 (*a*), and joined with a single fillet, the eccentricity tends to produce deformation in the members as shown in Fig. 28 (*b*). As this is an undesirable condition it is usually laid down that fillet welds shall not be used singly. Single fillets should never be used for carrying alternating stresses. The static strength of the single fillet depends to some extent on the relation between the plate thickness and the size of weld, but is about 70 to 80 per cent of the value of the fillet loaded without eccentricity as in

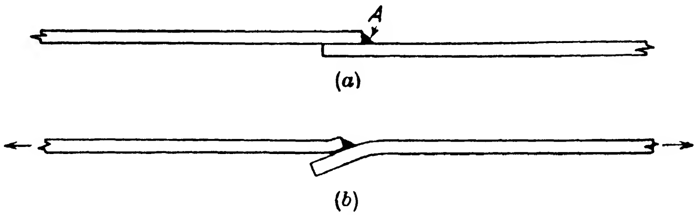


FIG. 28. TENSION BAR ATTACHED BY SINGLE END FILLET

Fig. 21. If deformation of the bars is to be avoided, the unit working stress in the bars must be kept very low.

The stress induced in the main plate in Fig. 21 by the two balanced fillets is evenly distributed and has the value P/t where P is the load per linear inch and t the thickness of the plate.

The stress in the plate near the toe of the single fillet at A in Fig. 28 (*a*) is made up of the direct stress P/t plus the stress due to the moment $P.a$ where a is the eccentricity of the forces, or the distance between the centre lines of the two plates. If the plates are of equal thickness, $a = t$. The stress at A is $P/t + Pa.6/t^2 = P/t + 6Pt/t^2 = 7P/t$, that is to say, the stress at the surface of the plate is seven times the stress in the centrally loaded bar. When deformation occurs under load, the moment arm is reduced by the bending of the bars.

(*b*) SECTION NOT SYMMETRICAL ABOUT AXIS. For the attachment of members such as angles, where the section is not symmetrical about the neutral axis, it is usual to set the neutral

axis of the member as nearly as possible on the line of stress, and to make the gravity axis of the group of welds forming the joint to coincide with the neutral axis of the member.

In the case of an angle attached to a gusset by side fillets, this would be achieved by balancing the length of the welds on the heel and toe of the angles so that $a_1 l_1 = a_2 l_2$ (Fig. 29 (*a*)) where l_1 and l_2 are the lengths of the welds on the toe and heel of the angle respectively and a_1 and a_2 are their distances

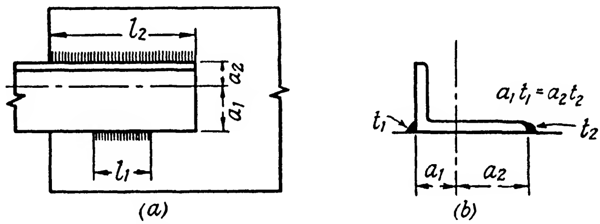


FIG. 29. ANGLE TENSION BAR ATTACHED BY SIDE FILLET WELDS

from the neutral axis. If the length of the welds is limited, the same effect may be achieved by making $a_1\ t_1 = a_2\ t_2$ where t_1 and t_2 are the throat thicknesses of the fillet at the toe and heel respectively (Fig. 29 (*b*)).

EXAMPLE 6. Angle connected by side welds. Two 3 in. × 3 in. × $\frac{1}{4}$ in. angle ties are connected to a gusset plate by means of side fillet welds.

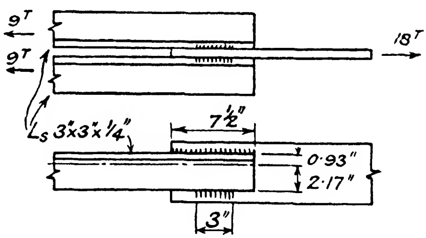


FIG. 30. ANGLE TENSION BARS ATTACHED BY SIDE FILLETS OF UNEQUAL LENGTHS. WELDS SHOWN ARE $\frac{1}{4}$-IN. FILLET WELDS

In Fig. 30 the load in each angle = 9 tons. From Table IV the allowable stress in $\frac{1}{4}$ in. side fillet welds = 0·9 ton per in.

The neutral axis of the angle is 0·83 in. from the heel. The load carried at the toe of the angle = 9 × 0·83/3 = 2·49 tons. Therefore the length of weld required = 2·49/0·9 = 2·75, say 3 in.

Load on weld at heel $= 9 \times 2{\cdot}17/3 = 6{\cdot}51$ tons.
Length of weld required
at the heel $= 6{\cdot}51/0{\cdot}9 = 7{\cdot}23$, say 7·5 in.

The example given illustrates the principle quite clearly, but it should be pointed out that this method of calculation is not accurate since it assumes that the distribution of stress is uniform throughout the section of the angle. It would be more consistent with fact to consider only 50 per cent of the outstanding leg as effective and proportion the welds on that basis. In most cases no serious error would arise if the two welds were made of equal length, and it has been claimed as a

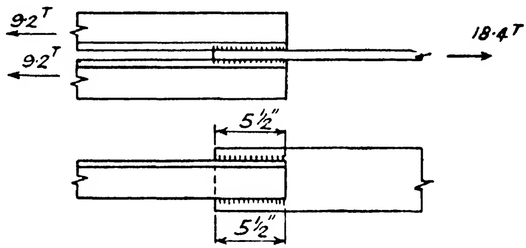


FIG. 31. ANGLE TENSION BARS ATTACHED BY SIDE FILLETS OF EQUAL LENGTH. WELDS SHOWN ARE ¼-IN. FILLET WELDS

result of experiment* that the stresses in the angle are actually less if the two welds are made equal.

If it is assumed that only one-half of the outstanding leg is effective, the area assumed to be carrying load is about 80 per cent of the full area of the angle.

EXAMPLE 7. Attachment of angle by side welds of equal length (Fig. 31). Two angle ties are to be attached on opposite sides of a plate gusset.

Area of 3 in. $\times$ 3 in. $\times$ ¼ in. angle $= 1{\cdot}44$ in.^{2}

Assuming a working stress of 8 tons per in.^{2}, the load in each angle $= 8 \times 1{\cdot}44 \times 80/100 = 9{\cdot}2$ tons.

From Table IV the working stress on ¼ in. side fillet $= 0{\cdot}9$ ton per in. Therefore length of weld required $= 9{\cdot}2/0{\cdot}9$ $= 10{\cdot}2$ in., say 5½ in. of weld on each side.

* Griffiths, Oregon State College, Oregon, U.S.A.

(c) SHEAR CONNECTIONS SUCH AS THAT BETWEEN THE FLANGE AND WEB IN PLATE GIRDERS OR BETWEEN THE PLATES AND FLANGES IN PLATED BEAMS.

EXAMPLE 8. A plate girder made up of a web plate 36 in. $\times \frac{3}{8}$ in. with flange plates 10 in. $\times \frac{3}{4}$ in. carries a uniformly distributed load of 104 tons on a span of 25 ft. (Fig. 32). It is

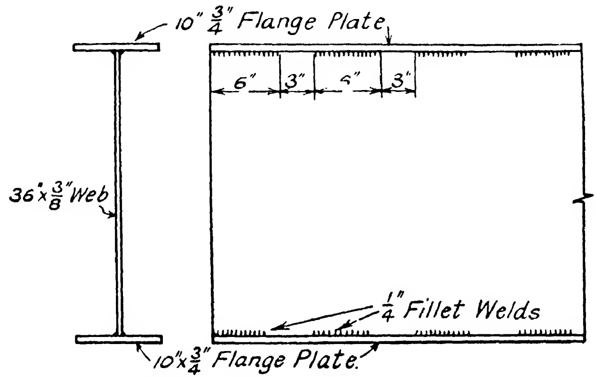


FIG. 32. WELDS JOINING FLANGE PLATES TO WEB IN PLATE GIRDER

required to find the size of weld necessary to join the flange plates to the web plates near the supports.

$$
\begin{aligned}
&\text{Moment of inertia of flange plates} &&= 5\,065 \text{ in.}^4\\
&\text{Moment of inertia of web} &&= 1\,458 \text{ in.}^4\\
&\text{Total moment of inertia of section} &&= 6\,523 \text{ in.}^4
\end{aligned}
$$

V = maximum shear (at end) $= 104/2 = 52$ tons.

Q = first moment of area of one flange

$= 10 \times \frac{3}{4} \times 18\frac{3}{8} = 137{\cdot}5 \text{ in.}^3$

Horizontal shear per inch $= VQ/I$

$= (52 \times 137{\cdot}5)/6\,523 = 1{\cdot}1$ tons.

From Table IV the strength of $\frac{1}{4}$ in. fillets is 0·9 ton per in. The length of weld required per foot

$= (1{\cdot}1 \times 12)/0{\cdot}9 = 14{\cdot}5$ in.

Welding 6 in. and missing 3 in. on each side gives 16 in. of weld per ft.

Joints Carrying a Combination of Stresses. When a weld is acted on simultaneously by two or more forces at right angles to each other the resultant stress may be taken as the vector sum of the two stresses (Fig. 33).

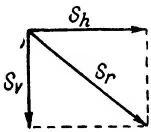


FIG. 33
VECTORIAL ADDITION OF STRESSES ON WELDS

$$S_r = \sqrt{(S_v^2 + S_h^2)}$$

where S_v = the stress in the weld due to the vertical load;

S_h = the stress in the weld due to the horizontal load;

S_r = resultant stress in the weld.

EXAMPLE 9. Welds stressed by two forces at right angles. An angle bracket stiffened by a flat plate carries a load of 10 tons at a distance of 3 in. from the line of the weld (Fig. 34).

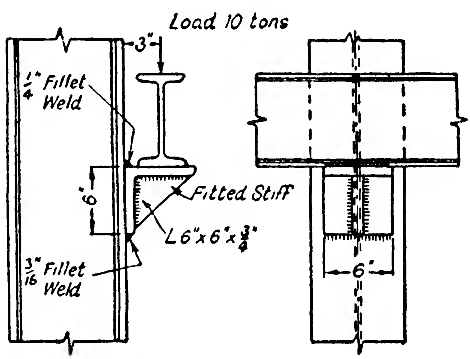


FIG. 34. END WELDS ON ANGLE BRACKET UNDER COMBINATION OF STRESSES

Assuming even distribution of stress along the length of the weld, the vertical and horizontal components of the stress on the weld are given by—

$$S_v = 10/12 = 0{\cdot}833 \qquad S_v^2 = 0{\cdot}694$$

$$S_h = (10 \times 3)/(6 \times 6) = 0{\cdot}833 \qquad S_h^2 = 0{\cdot}694$$

$$S_r^2 = S_v^2 + S_h^2 \qquad = 1{\cdot}388$$

Therefore S_r = 1·18 tons per in.

From Table IV a $\frac{1}{4}$ in. fillet as an end weld will carry a safe load of 1·2 tons per in. The $\frac{5}{16}$ in. fillet need only be used for the upper or tension weld. The horizontal thrust at the lower edge of the angle will be taken in direct bearing and only the vertical component 0·833 tons per in. need be provided for. For this a $\frac{3}{16}$ in. end fillet which will carry 0·9 ton per in. will suffice, though $\frac{1}{4}$ in. would probably be used.*

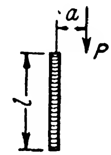


FIG. 35. SIDE FILLET WELD UNDER COMBINATION OF DIRECT STRESS AND BENDING MOMENT

Joints Carrying Direct Stress plus a Bending Moment in the Line of the Weld due to Eccentricity. (Fig. 35.)

A weld of length l carries a load P acting at a distance a from the line of the weld. The stress on the weld may be taken as the vector sum of the vertical and horizontal components.

Since the strength of fillets is usually reckoned per inch of length rather than per square inch of throat area it is possible to consider the weld as a line and use the linear dimension only. The vertical component of the force on the weld is given by—

$$S_v = P/l.$$

The horizontal component is given by—

$$S_h = (Pa \times 6)/l^2$$

where $P \times a$ = moment

and $l^2/6$ = section modulus of the weld.†

* A discussion of various methods of calculating combined stresses on fillet welds is given in the *Journal of the American Welding Society* for February, 1934, by Cyril D. Jensen of Lehigh University. The author points out that in his tests the results given by the method set out were always on the safe side up to as much as 37 per cent. It is therefore a simple though sometimes ultra-safe method and suitable for use in design. The method of calculation by principal stress fitted the experimental results a little more closely, but none of the methods of calculation was substantiated to the author's satisfaction.

† For the relation $M = f.I/y$ the moment of resistance of the fillet is given by $f.t \,.\, l^2/6$ where t is the throat thickness of the fillet; but the strength per linear inch of the fillet $= f.t$; therefore moment of resistance of the weld is given by (allowable stress per inch of weld) $\times\ l^2/6$.

The resultant stress is given by the vector sum of the vertical and horizontal components—

$$S_r = \sqrt{(S_v^2 + S_h^2)} = \sqrt{(P^2/l^2 + 36P^2a^2/l^4)}$$
$$= [P\sqrt{(l^2 + 36a^2)}]/l^2$$
$$= P \times C, \text{ where } C = [\sqrt{(l^2 + 36a^2)}]/l^2.$$

The term in brackets is a constant for any given case and it is therefore possible to prepare a chart giving the value of the constant C for various values of l and a.*

EXAMPLE 10. A bracket attached to a solid support by two welds 12 in. long carries a load of 12 tons at a distance of 3 in. from the line of the welds (Fig. 36).

For each weld—

$$V = P/l = 6/12 = 0{\cdot}5 \text{ ton per in.}$$
$$H = (P \times a \times 6)/l^2 = (6 \times 3 \times 6)/(12 \times 12)$$
$$= 0{\cdot}75 \text{ ton per in.}$$

The resultant stress—

$$S_r = \sqrt{(S_v^2 + S_h^2)} = \sqrt{(0{\cdot}5^2 + 0{\cdot}75^2)}$$
$$= \sqrt{0{\cdot}81} = 0{\cdot}9 \text{ ton per in.};$$

and, from Table IV, $\frac{1}{4}$ in. fillets (side weld) are more than sufficient.

Rigid Joints Transmitting Bending Stress in Addition to Direct Stress. The design of rigid joints usually involves two parts—the design of the welds to transmit the load between the parts of the joint, and the design of the parts themselves. The proportioning of the welds is, as a rule, relatively simple. The arrangement and design of the joint as a whole is frequently very complicated.

In general the moment of resistance of the group of welds forming the joint must be such that the impressed moments are carried without overstressing the welds. In the case of a

* Tabulated data for the design of fillet welds for eccentric brackets are given in the *Handbook of Welded Structural Steelwork* (Institute of Welding, 1398).

beam-to-stanchion joint it would appear to be necessary to make the moment of resistance of the group of welds equal to the moment of resistance of the beam or stanchion, whichever is the weaker. This constitutes a simple method of calculation but is frequently unduly conservative. Since the ultimate failure of a beam takes place at a stress not greatly above the yield point of the metal, it should be only necessary to provide sufficient strength in the welds to develop the yield point strength with the agreed factor of safety. In other words, if the real factor of safety on the member with relation to ultimate strength is only about 2·5 there is no need to provide a factor of safety of 4 on the welds.

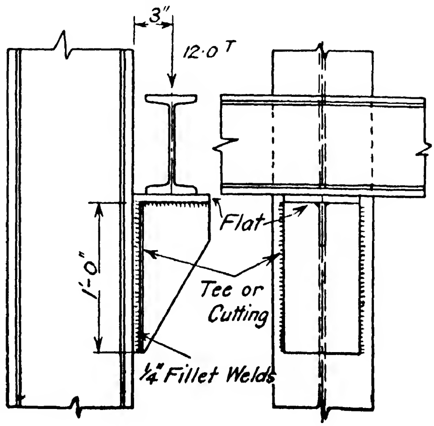


Fig. 36

Side Fillet Welds on Bracket under Combination of Shearing Force and Bending Moment

The major problem in designing rigid joints is in ensuring that the concentrated forces involved do not overstress the metal of the members themselves. It is frequently found that the weld is not at a point of maximum stress, and failure tends to take place right away from the welds.

Taking as an example two pieces of R.S. joist mitred and joined at right angles, if the loading is such that it tends to close the angle, failure occurs by buckling in the web opposite the ends of the inner flanges at a load much less than that required to develop the bending strength of the joists. If a plate is inserted between the two pieces, initial failure will probably occur by crippling in the added plate or, if the added plate is very stiff, in the web at some distance along the beam. If in this test the welds were made the full thickness of the

flange, the stress on the welds would never approach the yield point.

EXAMPLE 11. Two 12 in. × 5 in. × 30 lb. R.S.J.'s are to be connected on opposite sides of the web of a joist stanchion by fillet welds round the periphery of the beams (Fig. 37).

The moment of inertia of the 12 in. × 5 in. × 30 lb. R.S.J. is 206·9 in.^{4}

The moment of inertia of a continuous fillet weld $\frac{1}{2}$ in. fillet

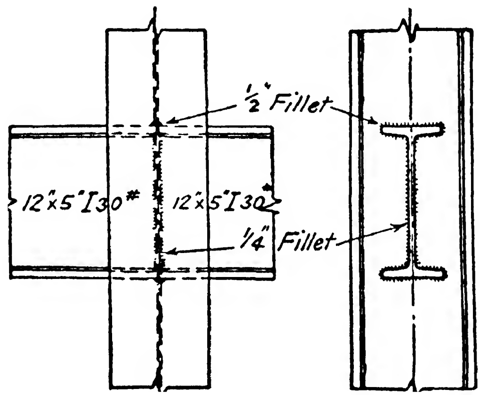


FIG. 37. JOIST CONNECTED BY FILLET WELD ROUND PERIPHERY

on flanges (throat = 0·354 in.) and $\frac{1}{4}$ in. fillet on web (throat = 0·176 in.) about the XX axis is as under—

Welds on web—

$(0{\cdot}176 \times 10^3 \times 2)/12$ . . . = 29·0.

Welds on flanges—

$(0{\cdot}354 \times 2) \times (5 \times 2) \times 5{\cdot}75^2$ . . = 234·0.

Total moments of inertia of welds . = 263·0 in.^{4}

The moment of resistance of the joist $= fI/y$

$= (8 \times 206{\cdot}9)/(6 \times 12) = 23$ ft. tons.

The moment of resistance of the weld $= fI_w/y$

$= (7 \times 263)/(6{\cdot}1 \times 12) = 25{\cdot}1$ ft. tons.

(From page 28, f_w for an end weld = 7 tons per in.^{2}) The

moment of resistance of the welds is thus somewhat greater than that of the joist, and the welds as assumed arc adequate.

EXAMPLE 12. Two 12 in. × 5 in. × 30 lb. R.S.J. beams are to be connected to the flanges of a 10 in. × 8 in. × 55 lb.

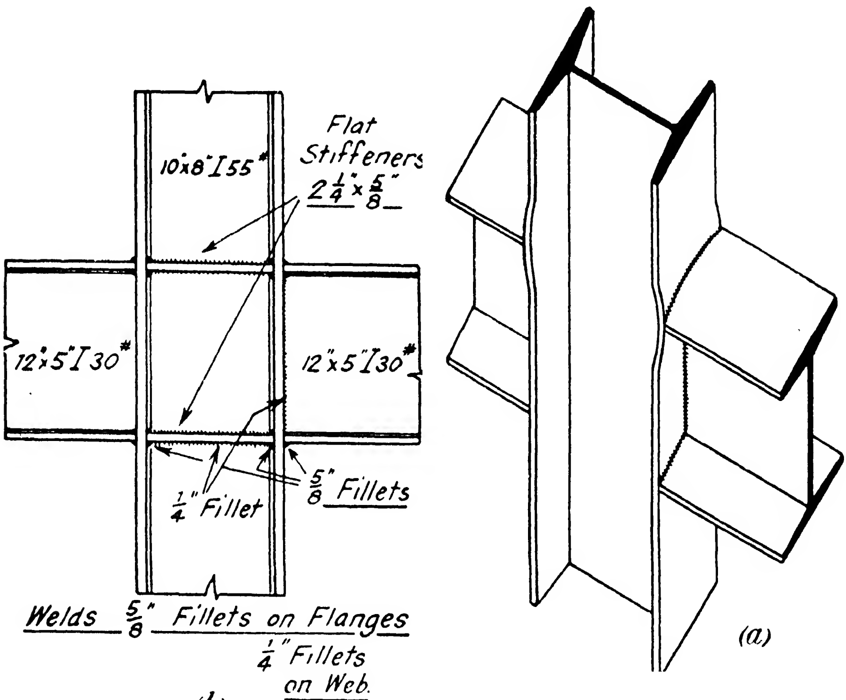


FIG. 38. RIGID BEAM-TO-COLUMN CONNECTION FOR INSIDE COLUMN

R.S.J. stanchion in such a way that a maximum moment is developed at the joints (Fig. 38). If the beam is designed for an allowable stress of 8 tons per in.^{2} there will be a tension of $8 \times 5 \times 0{\cdot}507 = 20{\cdot}3$ tons in the top flange ($0{\cdot}507$ = mean flange thickness) of the 12 in. × 5 in. × 30 lb. joist, and a compression of 20·3 tons in the bottom flange, and these two forces must be carried through the stanchion.

Assuming the load to be evenly distributed, the weld on the tension flange of the joist must be sufficient to carry 20·3/10 = 2·03 tons per linear inch and from Table IV a $\frac{1}{2}$ in. fillet, which will carry 2·5 tons per in., is required. It is, however, apparent that unless the flanges of the stanchion joist are stiffened in some way they will deform as indicated in Fig. 38 (*a*), and there will be a concentration of stress on the weld towards the middle of the flange. In order to keep the distribution of stress uniform, stiffening ribs must be welded in between the flanges of the stanchion joist to transfer the load through the stanchion (Fig. 38 (*b*)).

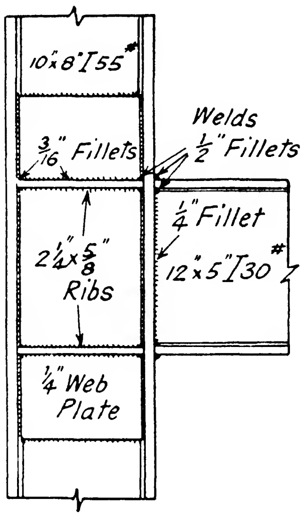


FIG. 39. RIGID BEAM-TO-COLUMN CONNECTION FOR OUTSIDE COLUMN

Showing full stiffening

In order to avoid introducing large transverse tensions into the web of the stanchion it is desirable that the straps should be capable of carrying the full tension load. Each strap would carry 20·3/2 = 10·15 tons and for this $2\frac{1}{4}$in. × $\frac{5}{8}$ in. flat would suffice.

On the compression side a proportion of the load may be considered as carried through the web but the working stress in the straps must be reduced. If it is assumed that the load from 1 in. of weld is carried through the web, the load to be carried by each strap is (20·3 × 2)/5 = 8·12 tons. Assuming a working stress of 6·5 tons per in.^{2}, an area of 8·12/6·5 = 1·25 in.^{2} would be required. Both straps would therefore be made $2\frac{1}{4}$ in. × $\frac{5}{8}$ in. The weld at each end of the straps should be sufficient to carry 10·15 tons or 10·15/(2·25 × 2) = 2·34 tons per in.: requiring a $\frac{5}{8}$ in. end fillet. The straps would be attached to the web with a single-run weld $\frac{3}{16}$ in. or $\frac{1}{4}$ in.

EXAMPLE 13. A 12 in. × 5in. × 30 lb. R.S.J. beam is to be

connected rigidly to a 10 in. × 8 in. × 55 lb. R.S.J. stanchion, transferring the full negative moment to the stanchion (Fig. 39).

As in Example 12 above there will be a tension of $8 \times 5 \times 0{\cdot}507 = 20{\cdot}3$ tons in the top flange and a compression of 20·3 tons in the bottom flange, and these two loads must be transferred to the stanchion section without overstressing the metal.

A point load of 20·3 tons would stress the web in shear to $20{\cdot}3/(0{\cdot}4 \times 8) = 6{\cdot}34$ tons in.^{2} (0·4 in. = thickness of stanchion web; 8 in. = depth of web).

The allowable buckling stress —

$$5{\cdot}5 - 0{\cdot}04\,(d/t) = 5{\cdot}5 - 0{\cdot}04\,(8/{\cdot}40)$$
$$= 4{\cdot}7 \text{ tons per in.}^2$$

It is therefore necessary to stiffen the web of the stanchion for shear and buckling. To give the necessary strength, the web of the stanchion is strengthened by the addition of two $\frac{1}{4}$ in. web cover plates welded solidly to the flanges (Fig. 40). The shear in the web is then reduced to

$$20{\cdot}3/(0{\cdot}4 + 0{\cdot}5) \times 8$$
$$= 2{\cdot}82 \text{ tons per in.}^2$$

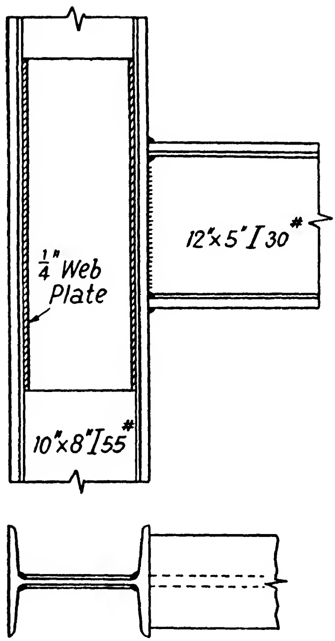


FIG. 40. RIGID BEAM-TO-COLUMN CONNECTION FOR OUTSIDE COLUMN
Showing stiffening of web of column

As in Example 12, it is necessary to add stiffening ribs to support the flange and to transfer the load to the web of the stanchion. If the web and the added plates are considered to carry the load from the middle inch, each rib would be required to carry a load $20{\cdot}3 \times 2/5 = 8{\cdot}12$ tons and, for this,

flats $2\frac{1}{4}$ in. × $\frac{5}{8}$ in. are required. The welding on the flat stiffeners must be sufficient to pick up the load of 8·12 tons from the flange and transfer it to the web. For the inner end of the stiffener the stress on the weld will be 8·12/4 = 2·03 tons per in. as an end weld which from Table IV requires a $\frac{1}{2}$ in. fillet. For the attachment to the web the working stress on the weld is 8·12/16 = 0·51 ton per in. as a side weld for which a $\frac{3}{16}$ in. fillet suffices (Fig. 39).

If the web of the stanchion is sufficiently strong to carry

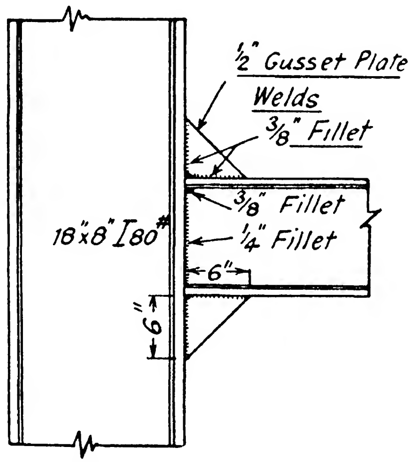


Fig. 41. Alternative Rigid Beam-to-column Connection for Small Beam

the shear from the beam flanges, flat triangular gussets may be added above and below the beam in the plane of the web to transfer the load in the flanges direct to the webs.

Example 14. A 12 in. × 5 in. × 30 lb. R.S.J. beam is to be attached to an 18 in. × 8 in. × 80 lb. R.S.J. stanchion (Fig. 41). The load in each flange of the beam is 8 × 5 × 0·507 = 20·3 tons. It might be reasonable to assume that $\frac{1}{3}$ of the load or 20·3/3 = 6·8 tons is transferred by the welds on the web and flanges of the joist and that the remaining $\frac{2}{3}$ or 13·5 tons are carried by the gussets. In a 6 in. × 6 in. × $\frac{1}{2}$ in.

gusset, the area along the diagonal from the right angle is $6 \times 0{\cdot}7 \times 0{\cdot}5 = 2{\cdot}1$ in.^{2} and the stress in the gusset $13{\cdot}5/2{\cdot}1 = 6{\cdot}42$ tons per in.^{2} Since it is desirable to develop the full strength of the gusset, the weld is required to carry a load of $2{\cdot}1 \times 8 = 16{\cdot}8$ tons. The load on the weld is thus $16{\cdot}8/12 = 1{\cdot}40$ tons per in., requiring a $\frac{1}{2}$ in. fillet. The weld between the flanges and the upper part of the web of the beam and of the stanchion must be sufficient to carry the remainder of the load,

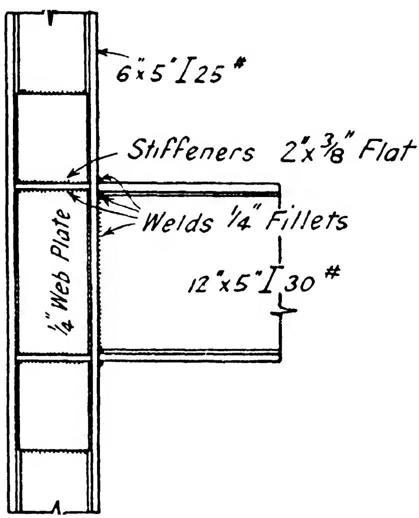


FIG. 42. RIGID BEAM-TO-COLUMN CONNECTION FOR HEAVY BEAM TO LIGHT COLUMN

i.e. $6{\cdot}8$ tons. This is readily provided for by a $\frac{3}{8}$ in. fillet. In this case it is found that the web of the stanchion is sufficiently strong to carry the impressed loads. Area of web $= 16 \times 0{\cdot}5 = 8{\cdot}0$ in.^{2} The shear on the web $= 20{\cdot}3/8 = 2{\cdot}54$ tons per in.^{2}

The load may be assumed to be spread over a length of 12 in. of web and the buckling stress is therefore $20{\cdot}3/(12 \times 0{\cdot}5) = 3{\cdot}4$ tons per in.^{2} This allowable buckling stress $= 5{\cdot}5 - 0{\cdot}04(16/0{\cdot}5) = 4{\cdot}22$ tons per in.^{2}, and therefore no stiffeners are required.

When the beam is substantially stronger than the stanchion it is sufficient to develop the strength of the smaller section.

EXAMPLE 15. Connection of 12 in. × 5 in. × 30 lb. R.S.J. beam to 6 in. × 5 in. × 25 lb. R.S.J. stanchion (Fig. 42).

Modulus of Section of 6 in. × 5 in. × 25 lb. R.S.J. = 15·07 in.^{3}

Therefore moment of resistance at 8 tons per in.^{2}

$$= (15{\cdot}07 \times 8)/12 = 10 \text{ ft. tons.}$$

Therefore the maximum negative bending moment which can be introduced into the beam is 10 ft. tons. Ignoring the weld on the web of the joist, the welds on the flanges will be required to carry a load of $(10 \times 12)/11{\cdot}5 = 10{\cdot}4$ tons (distance between centres of gravity of welds on flanges = 11·5 in.) or $10{\cdot}4/10 = 1{\cdot}04$ tons per linear inch of weld. If the joist is welded right round with a $\frac{1}{4}$ in. fillet there will be an ample margin of strength. It is necessary to reinforce the web of the 6 in. × 5 in. × 25 lb. stanchion with additional web plate and stiffeners as in Example 13.

CHAPTER VI

DESIGN OF STRUCTURAL UNITS

Roof Trusses. The triangulated roof truss, in which the loads in all members are in simple tension or compression, introduces the steel frame in its simplest form.

EXAMPLE 16. 21 ft. span Fink type roof truss.

The shop drawing for a very light Fink type roof truss of 21 ft. span is shown in Fig. 43. No stress diagram is given because the size of the members is not determined by the loads, as they are the minimum sizes which it seems reasonable to use. The joints are made sufficiently strong to develop the full section of the members. The chords of 3 in. $\times$ 2 in. $\times$ $\frac{1}{4}$ in. angle and web members of 2 in. $\times$ $1\frac{1}{2}$ in. $\times$ $\frac{1}{8}$ in. angle are quite strong enough for the small loads they have to carry when the members are not reduced by punching rivet holes and when the amount of welding used is sufficient to develop the full strength of the members, for the truss would carry a distributed load of 10 tons without failure.

The simplicity of the detailing is noticeable. Gauge lines are discarded and all measurements are taken from the backs of the angles. In the members which are most heavily stressed the centroid of the section is placed as nearly as possible on the line of stress. The position of the welds is shown by hatching on full-size details of the joints. The welds are specified 10/9, i.e. one run of No. 10 gauge, laying down 9 in. of weld per electrode. This weld is approximately a $\frac{3}{16}$ in. fillet.

In Table IV the working stress for a $\frac{3}{16}$ in. fillet as an end weld is given as 0·9 ton per in. or as side weld 0·6 ton per in. Considering the joint KQ- KD, the angle KD (2 in. $\times$ $1\frac{1}{2}$ in. $\times$ $\frac{1}{8}$ in.) is capable of carrying a safe load of $0{\cdot}3 \times 8 = 2{\cdot}4$ tons and therefore requires $2{\cdot}4/0{\cdot}9 = 3$ in. of end weld or $2{\cdot}4/0{\cdot}6 = 4$ in. of side weld.

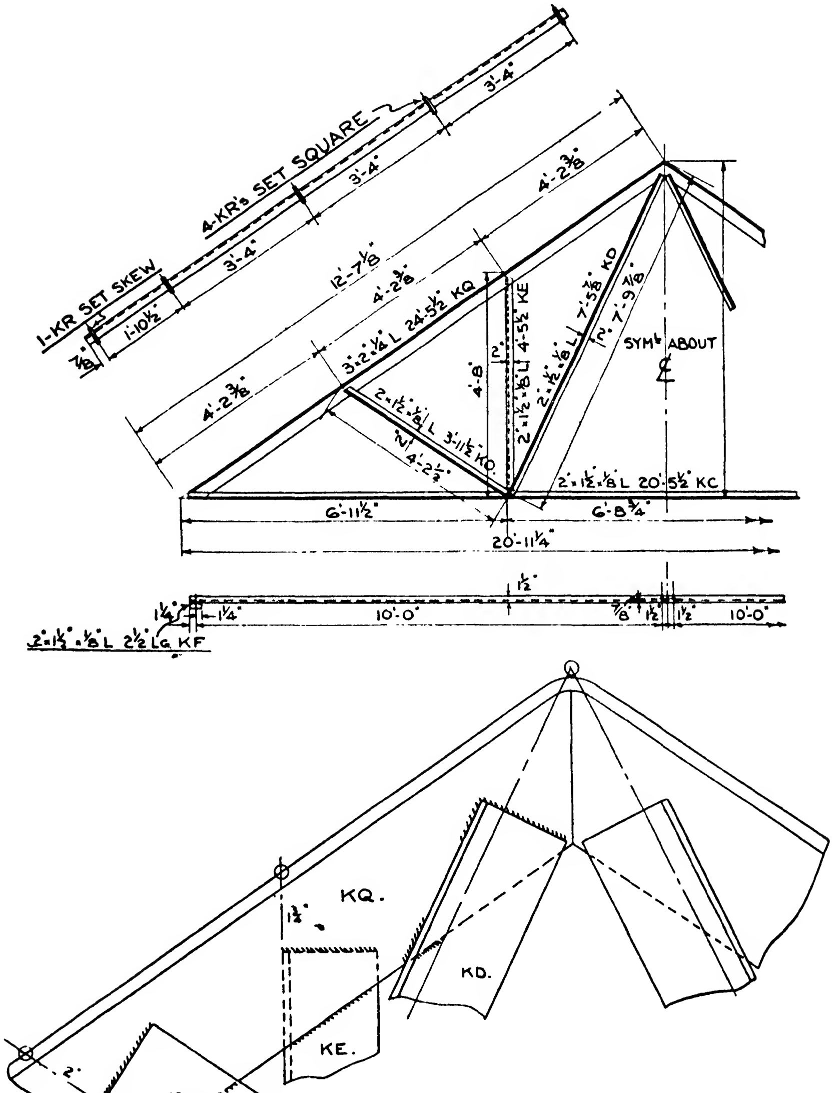
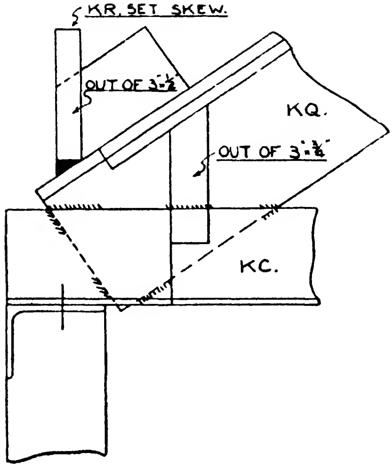
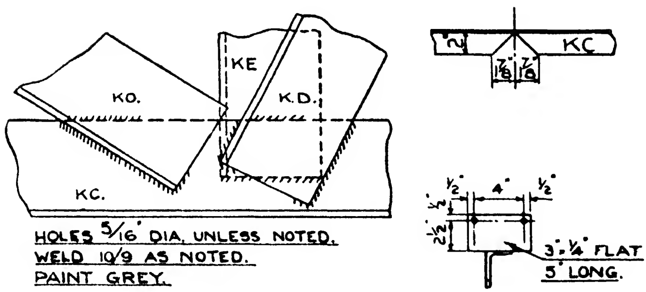


Fig. 43. Fink Type Roof





Truss 20 ft. Span

3—(T.31)

The weld consists of—

$$\text{end weld } 2 \times 0{\cdot}9 = 1{\cdot}8$$
$$\text{side weld } 4 \times 0{\cdot}6 = 2{\cdot}4$$
$$\text{Total} = 4{\cdot}2 \text{ tons.}$$

Thus the full strength of the angle is developed by the weld. It will be noted that there are no gussets and that the purlin cleats consist of flat plates on edge welded to the rafter.

EXAMPLE 17. 40 ft. span Fink type roof truss.

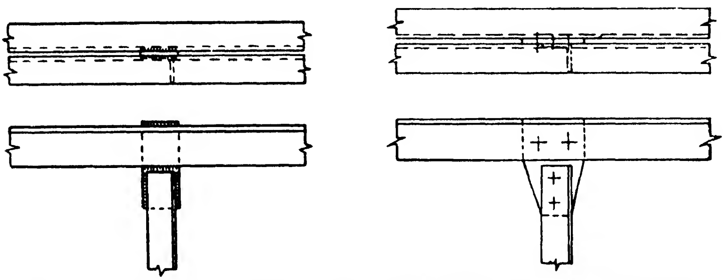


FIG. 46. WELDED AND RIVETED ATTACHMENT OF GUSSET PLATE

Stress diagrams for dead load and wind load and schedule of loads and working stresses in the members are given in Fig. 44 for a 40 ft. span Fink type truss. The arrangement of the joints and welding details are given in Fig. 45.

For spans up to about 40 ft. the truss may be made with single members throughout and gussets can be almost entirely eliminated. When the span is greater than about 40 ft. the choice of a section for the rafter requires consideration. In a riveted truss the rafter would normally be made with two angles connected by gussets. This arrangement is not economical in welded construction because the use of a gusset involves doubling the amount of welding at the joint. It will be seen that in Fig. 46 the one rivet in the rafter member would connect both angles to the gusset, whereas each welded member must be connected separately.

In welded construction it is desirable and generally possible to devise an arrangement whereby web members are connected

directly to the chord members without gussets. For trusses up to 40 ft. span the rafter may consist of a single rolled angle or tee, but for greater spans no suitable single section is rolled. A tee section would involve less eccentricity than the single angle, but with standard tees the outstanding leg is too short to allow effective joints to be made, and the tee rafter is deficient in

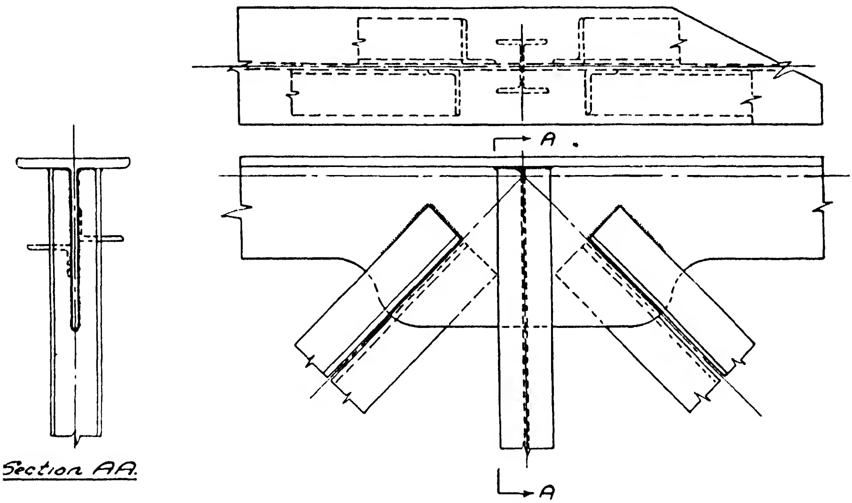


FIG. 47. ATTACHMENT OF WEB MEMBERS TO SHAPED-TEE CHORD SECTION CUT FROM R.X.J.

bending strength. A deep tee section may be made by splitting an R.S.J. The length of the outstanding leg then enables effective connections to be made, but the section is expensive in that it involves the splitting operation. In Fig. 47, detail is given in which the web is shaped by gas cutting to give extra depth where it is required for the connections.

In Fig. 48 the top chord consists of two channels set back to back and spaced sufficiently far apart to allow the web members to be threaded between them and connected to them direct without any gussets. One leg of the web angle is connected flat against one channel, while the outstanding toe is welded to the other channel. If any of the web angles is of greater distance

than the distance between the channels, it is made to fit by notching the outstanding leg. The 2 in. space used in this case allows ample room for making good welds and the centroid lines of all the members are placed as nearly as possible on the lines of stress. A maximum of rigidity both vertically and laterally is achieved with a minimum of weight.

EXAMPLE 18. Fink type roof truss, 69 ft. span.

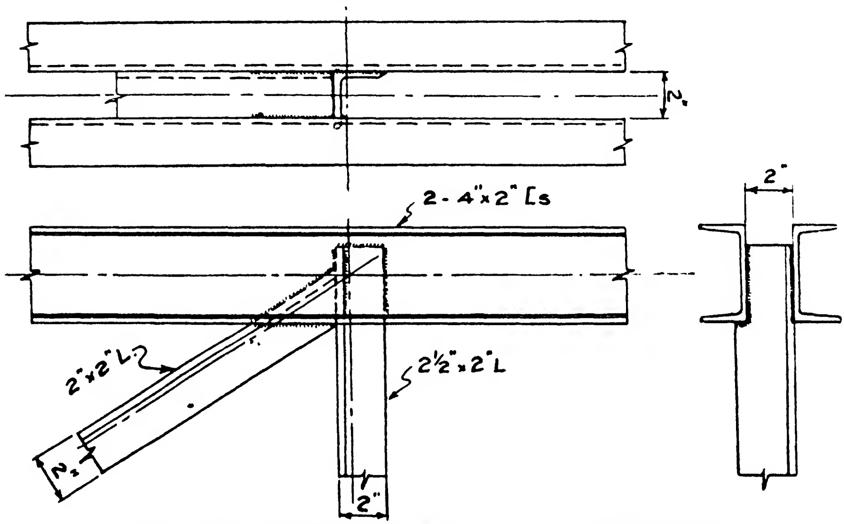


FIG. 48. ATTACHMENT OF WEB MEMBERS TO DOUBLE CHANNEL CHORD MEMBER

Working drawings for a Fink type roof truss of 69 ft. span are given in Fig. 49. The rafter and tie consist of two channels set 2 in. apart with the web members between them. The arrangement at the joint is neat and simple. It will be seen that the only gusset is at the apex of the truss and is set perpendicular to the plane of the truss.

At the abutments the top chord channels are cut on the bevel and welded to the top of the bottom chord channels which run right through. The thrust from the flanges of the top chord channels is carried through the lower chord channels by flat stiffening pieces. The holes for the holding down bolts are

through the bearing plate. The truss is fabricated in pieces and erected with temporary bolts. The joint in the bottom chord is completed with a field weld. In the tension members, spacing pieces of flat bar perpendicular to the plane of the truss are inserted at appropriate intervals (Fig. 50A). In the compression members the spacing pieces must be arranged so that they may act as battens to make the two members of the strut act as a unit, and must be capable of transmitting shear between the two members. In the detail shown the purlin cleats act as spacing pieces and battens. Various possible arrangements of the detail are shown in Figs. 50B and 50C.

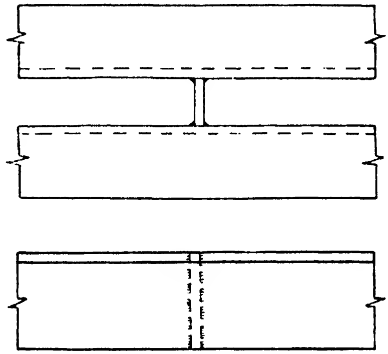


FIG. 50A. SPACING PIECES FOR DOUBLE-ANGLE TENSION MEMBERS

EXAMPLE 19. Steelwork for Mill Building with 100 ft. span lattice trusses and 30 ft. span north light roof truss.

The general arrangement of the steelwork for a mill building 100 ft. by 262 ft. without internal columns is shown in Fig. 51. The main lattice girders span across the building and are designed to take the transverse wind load. The roof trusses are carried on the top and bottom booms of the main girders and the longitudinal wind drag is transmitted through the rafters and ties of the roof trusses to the horizontal wind girders in the second and sixth bays, whence it is carried to the foundations through the vertical bracing.

The loads and stresses in the members of the roof trusses are set out in Table V, and detail drawings for the truss are given on Fig. 52.

Allowance is made in the rafter and tie for the additional load due to the longitudinal wind and, in the case of some of the trusses in the second and sixth bay, this has the effect of reversing the load in the tie member which is therefore designed

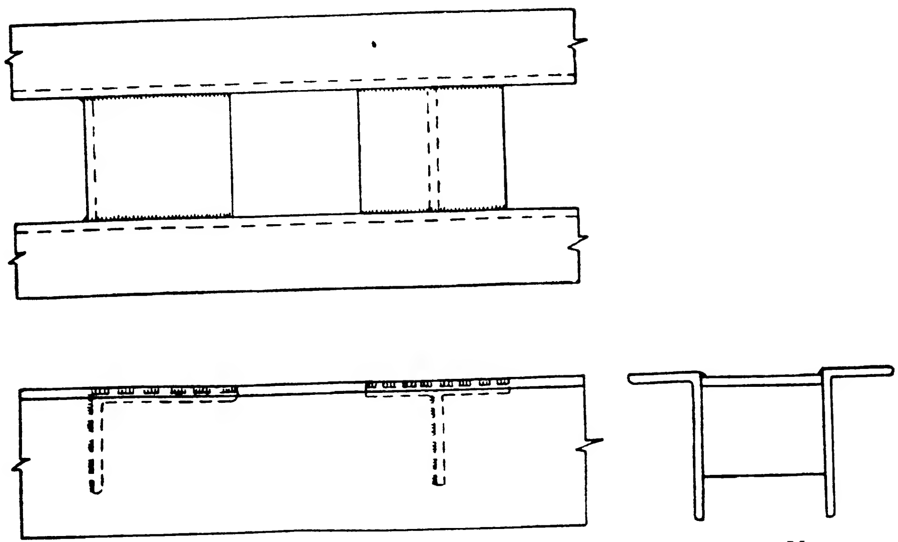


Fig. 50b. Spacing Pieces and Battens for Double-angle Compression Members

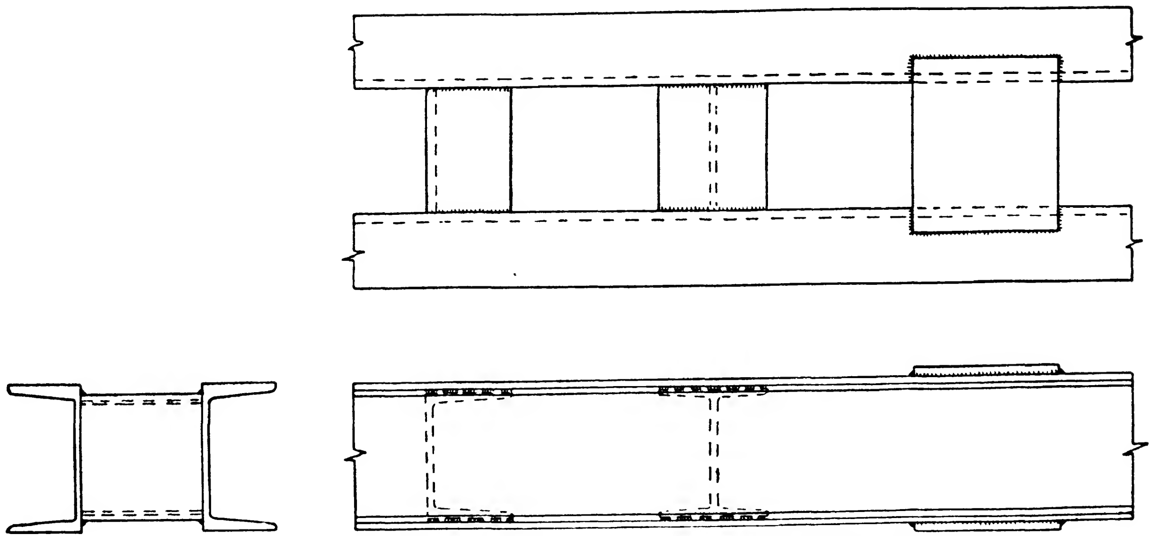


Fig. 50c. Spacing Pieces and Battens for Double-angle Compression Members

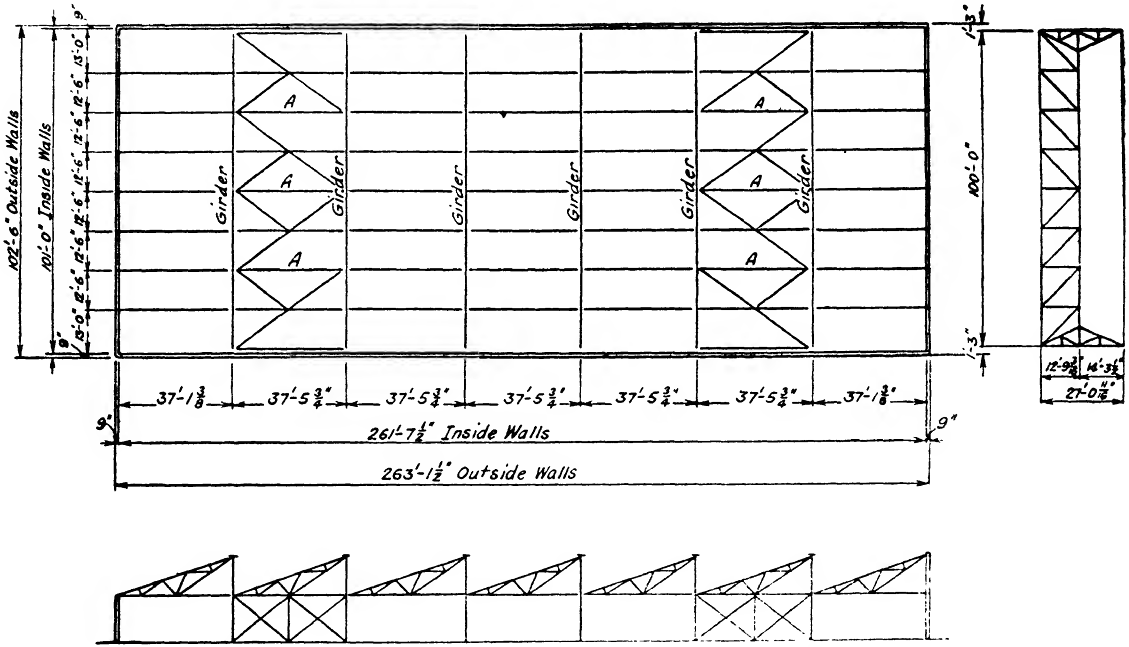


FIG. 51. GENERAL ARRANGEMENT OF STEELWORK FOR MILL BUILDING

as a strut. This variation is also shown. The lattice girders and knee bracing are considered as a unit and designed as a two-pin frame. The joint details are given in Fig. 53. The top and bottom booms consist of two angles set apart to allow the

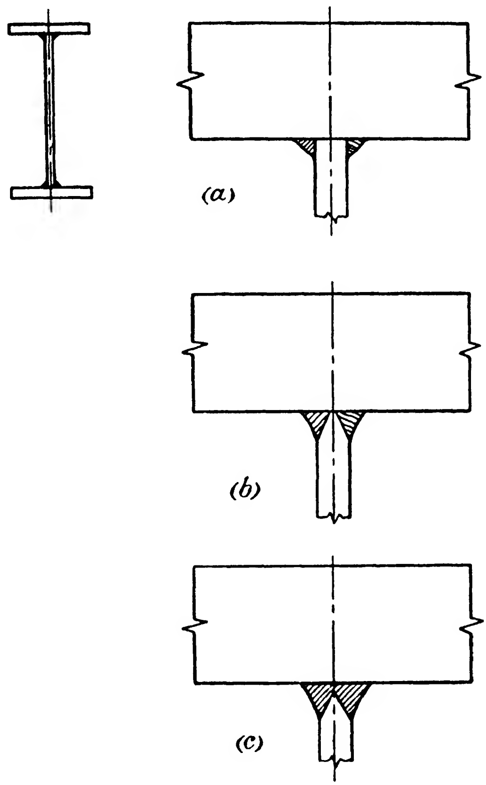


FIG. 54. ATTACHMENT OF FLANGES TO WEB PLATE

vertical struts to be let in between them. The diagonal ties are attached on the outsides of the chord angles and excellent connections are obtained with a complete absence of gussets. The main frames are welded up complete before erection. The roof trusses are attached to the girders by bolted connections,

but it will be seen that no holes are made in the main members of the girders.

Plate Girders. The conventional riveted assembly for plate girders consisting of a web plate with four angles and flange plates is not satisfactory for welded construction. The most simple arrangement consists of a web plate with flange plates welded to it direct by fillet welds. For small girders a channel section may be used for the compression flange to provide increased lateral stiffness.

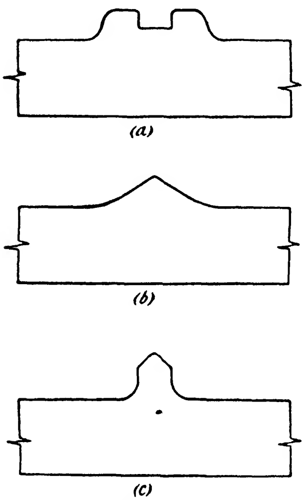


FIG. 55. SPECIAL SECTIONS FOR FLANGES OF PLATE GIRDERS

The connection between the web and flange plates is normally made as in Fig. 54 (*a*), with two fillets of welding. The calculation for this weld is given on page 43. For girders of normal shape the fillet welds required are quite small. When the shear is heavy and large welds are required, it is sometimes economical to chamfer the edges of the web plate, and so obtain the necessary throat thickness with a smaller quantity of weld metal (Fig. 54 (*b*)).

Special sections used for the flange plates of girders are shown in Fig. 55. In type (*a*) the provision of ridge and slot facilitates the assembly of the girder, and any tendency to distortion of the flange by folding about the line of weld is restricted to the ridge and does not affect the main part of the plate.

Types (*b*) and (*c*) enable the flanges to be joined to the web by butt welds and the shape of the resultant joint avoids the concentrations of stress which are inherent in a fillet weld. It is used particularly for railway bridge work and structures subjected to dynamic loading. A similar effect is obtained, when using a flat plate as flange, by chamfering the edges of the

web plate to a V as in Fig. 54 (*c*), but this joint presents the difficulty of maintaining an effective gap, for the plate tends to be drawn in between the tack welds, and in building up the fillets to a size which gives a gradual change of section, distortion of the flange plate by folding about the line of the weld is likely to occur.

WEB STIFFENERS. Web stiffeners usually consist of flats

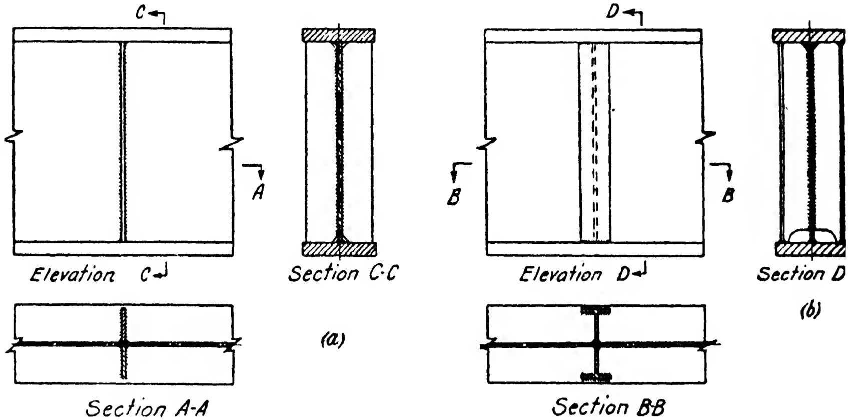


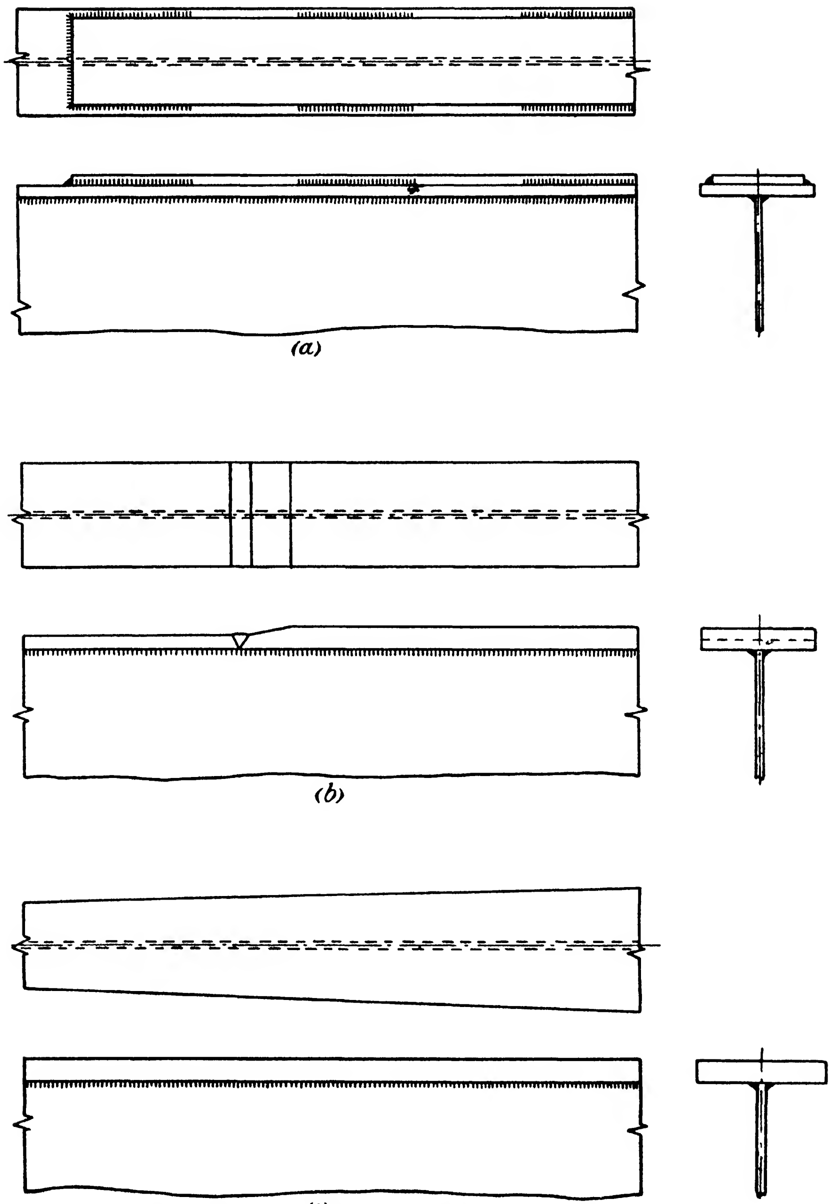
FIG. 56. STIFFENERS FOR PLATE GIRDER WEBS

welded on edge to the web plate (Fig. 56 (*a*)). In addition to the usual functions of supporting a point load or of stiffening the web plate against buckling under the shear load, the stiffeners on a welded plate girder are used partly to maintain the shape of the girder, because the connection at the junction of flange and web is relatively more flexible than the riveted arrangement with angles, and any tendency for the flange to rotate would induce secondary stresses of unknown amount in the fillets. For a stiffener carrying load the thickness should be at least one-twelfth of the width. The stiffener should be placed directly under the load and close fitted or welded to the compression flange. The amount of welding must be sufficient to transfer the load to the web. For a stiffener to resist web

buckling only, the thickness should not be less than one-sixteenth of the width. The stiffeners are welded to the web with continuous or intermittent fillets. The longitudinal distance between welds should not be more than sixteen times the thickness of the web plate or stiffener. Opinion differs as to the desirability of welding the ends of the stiffeners to the flanges. If the function of the stiffener is to strengthen the web plate against buckling, there is no need to weld the stiffener to the flanges, but it is usually welded to the top flange to keep it square, and frequently also to the bottom flange. When the stiffeners are spaced far apart, small triangular gussets between the web and the flange plates may be used to preserve the shape of the girder and to avoid torsional stresses on the welds. If the flange plates are light, the fillets joining the stiffeners to the flanges must be very small to avoid distorting the flanges.

If the girder is to be subjected to alternating stresses or dynamic stresses of high magnitude it is preferable to avoid welding the stiffener to the bottom flange, or at least to avoid placing the weld across the flange. If a weld is made across the lines of stress on a bar carrying an alternating load, the concentration of stress at the edges of the weld due to the change of section at the point is sufficient to induce failure at a load very much less than that which the plain bar would carry. By using a tee or angle stiffener connected to the web by the toe it is possible to obtain a very effective arrangement as indicated in Fig. 56 (*b*). The stiffening effect is much greater than with a flat plate and, since the weld on the flange is placed along the lines of stress, the concentration of stress is avoided or reduced to negligible proportions. If the leg of the tee which is joined to the web is cut away, the effect is good and there is no obstruction to the making of the fillet welds joining the flanges to the web.

Flanges. The flange area may be varied to conform to the bending moment diagram, by adding flange plates as in Fig. 57 (*a*), or by using plates of different thickness joined by



(a)

(b)

(c)

Fig. 57. Methods of Varying Sectional Area of Flanges

butt welds as in Fig. 57 (*b*), or by varying the width of the plates as in Fig. 57 (*c*). When additional flange plates are used, they are made wider or narrower than the main flange plates in order to allow the fillet welds to be made. It is usual to provide sufficient weld to develop the effective strength of the plate within a distance from the end equal to the width of the plate. It is then only necessary to carry the plate beyond

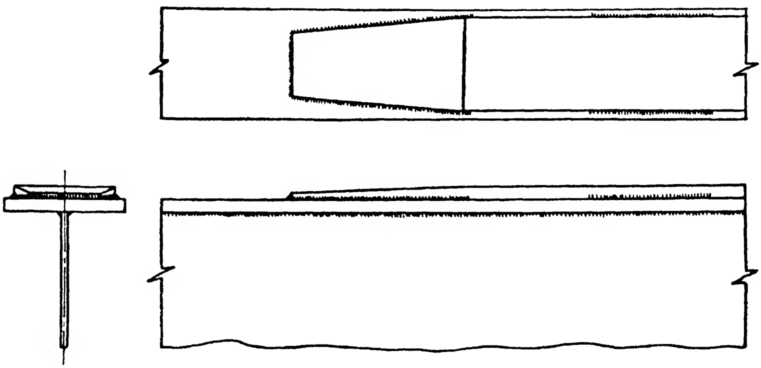


FIG. 58. ATTACHMENT OF ADDED FLANGE PLATES

the point where it is theoretically required to the extent of a few inches.

For a girder subjected to alternating stress the abrupt change of section at the square end of an added flange plate would cause a serious concentration of stress. The effect may be minimized by tapering the added flange plate either in width or thickness or both, and extending it beyond the length theoretically required (Fig. 58).

When the flange area is varied by using plates of different thickness it is advantageous to make the butt welds before the flanges are joined to the web so that the shrinkage across the butt weld may take place freely without inducing locked-up stresses. When the thickness of the plates joined by the butt weld varies by more than $\frac{1}{4}$ in., the thicker plate should be

chamfered to a bevel of not more than one in four, to the thickness of the thinner plate.

To counteract the tendency for a concentration of stress to exist at the ends of butt welds in wide plates, the outside edges of the butt weld in the tension flange may be reinforced with cover straps as in Fig. 59. Since the cover straps are only intended to relieve the concentration of stress at the ends of the

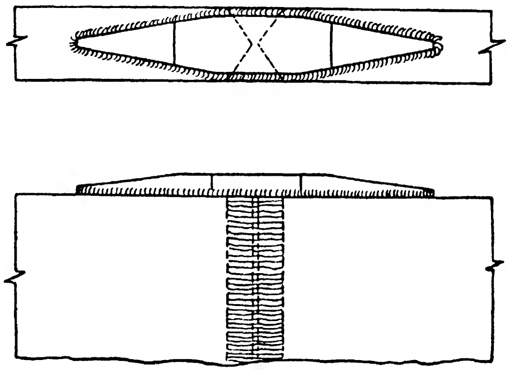


FIG. 59. COVER STRAPS FOR REINFORCING ENDS OF BUTT WELDS

welds, and are not considered as reinforcing the butt weld, they may be made quite small. Each strap need only be 2 to 3 per cent of the area of the plate. In order to be effective the strap must be designed in such a way that under ultimate load it will fracture at the centre before the ultimate strength of the welds is reached. To effect this the ends of the straps must be tapered in two directions as shown. If the ends of the strap are left square the welds start to tear from the ends when the yield point of the main bar is reached.

Arrangements which have been used to obviate the use of cover straps by increasing the length of the butt weld are indicated in Fig. 60. The diagonal butt weld of Fig. 60 (*a*) shows a marked improvement in fatigue strength when compared to the butt weld at right angles. The vee arrangement

of Fig. 60 (*b*) is not as good because of the difficulty of making the weld perfectly at the point and the possibility of stress concentration at the point.

It is now more usual to carry the weld beyond the edges of the flange plates by welding on temporary plates to give the shape of the vee, the added plates being removed on completion of the weld, and the end of the weld chipped or ground smooth. By this means the bad ends of the welds are removed, and there is no discontinuity or irregularity at the end of the weld.

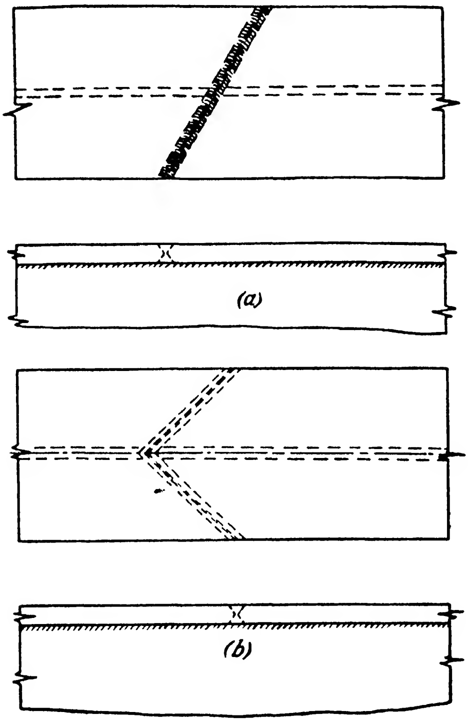


FIG. 60. BUTT WELDS PLACED DIAGONALLY ACROSS FLANGES TO REDUCE EFFECT OF STRESS CONCENTRATION

EXAMPLE 20. A girder, 36 ft. span between centres of supports, carries a point load of 90 tons, 12 ft. from one end support, and a uniformly distributed load of 2 tons per ft. run of girder, which includes the weight of the girder.

The reactions are (Fig. 61)—

$$R_1 = 36 + \frac{90 \times 24}{36} = 36 + 60 = 96 \text{ tons.}$$

$$R_2 = 36 + \frac{90 \times 12}{36} = 36 + 30 = 66 \text{ tons.}$$

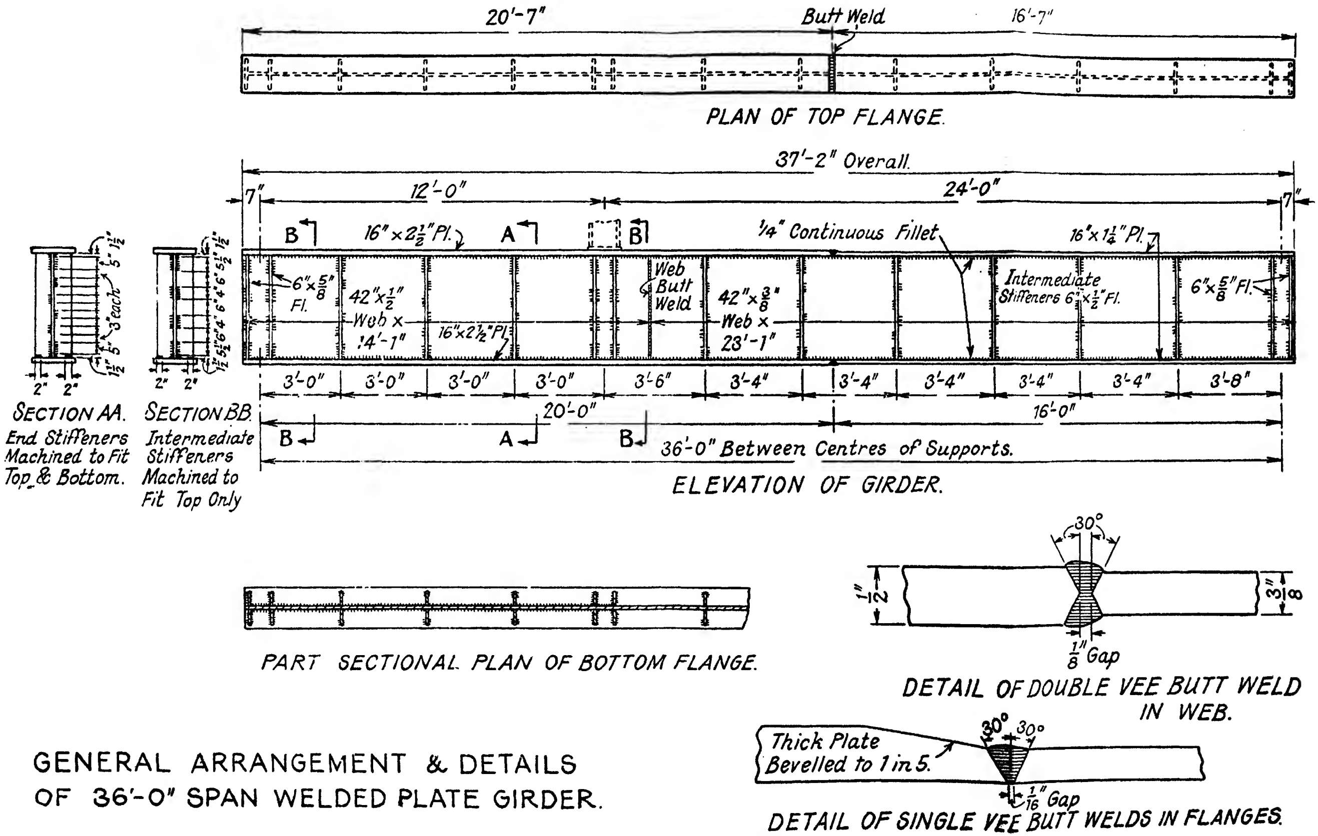


Fig. 61. 36-ft. Span Plate Girder

The maximum bending moment due to the uniform load is $\dfrac{72 \times 36}{8} = 324$ ft.-tons.

The bending moment due to the point load at the point of application is $60 \times 12 = 720$ ft.-tons. It is assumed that the girder is supported laterally, and the allowable stress for both compression and tension flanges is taken as 8 tons per in.^{2} The allowable shear stress in the web is taken as 5 tons.

The maximum bending moment at the point of application of the load, including the uniform load, is

$$(96 \times 12) - (2 \times 12 \times 6) = 1\,152 - 144 = 1\,008 \text{ ft.-tons.}$$

The depth of the web is taken as 42 in. Between the point load and the left-hand end of the girder, the maximum shear is 96 tons, for which a $\frac{1}{2}$ in. web may be used.

Between the point load and the right-hand end the maximum shear is 66 tons, and a $\frac{3}{8}$ in. web is ample.

To carry the maximum bending moment under the point load, the flanges are made 16 in. $\times$ $2\frac{1}{4}$ in. At 16 ft. from the right-hand end the flange may be reduced to 16 in. $\times$ $1\frac{1}{4}$ in. The web is stiffened by flat stiffeners at the supports and under the point load to carry the full shear at these points, and at intermediate points along the girder to carry a proportion of the vertical shear calculated by the formula. (*B.S.S.* 153, Part 4, 13.)

$$s = \tfrac{1}{4}S(p/D),$$

where s = vertical shear for which each stiffener or pair of stiffeners is to be designed;

S = maximum vertical shear at the position attachment;

D = overall depth of girder;

$p =$ the sum of the distance between the centre line of the stiffener or pair of stiffeners in question and the centre lines of the adjacent stiffeners.

Flange Butt Welds. The allowable working stress in the butt welds in the flanges is 8 tons per in.^{2}, the same as for the flange plate. The joint between the different thicknesses of plate is made with a simple butt weld, either single or double V, or single or double U. As the surfaces of the two plates are more than $\frac{1}{4}$ in. out of line, it is necessary to bevel the end of the thicker plate.

Web Butt Welds. The butt weld in the web is placed 18 in. from the point of application of the concentrated load. Since the allowable shear stress on the weld is 5 tons per in.^{2} and the design stress on the web is less than 5 tons per in.^{2}, a simple butt weld suffices.

Weld Joining Flanges to Web. As referred to in Chapter V (page 43), the longitudinal shear per inch run between the flanges and web is given by the formula

$$s = VQ/I.$$

Near the left-hand end support the shear is equal to the end reaction, 96 tons. The first moment $Q = 16 \times 2\frac{1}{4} \times 22{\cdot}12 = 800$ in.^{3}, and the moment of inertia of the section equals

$$s = \frac{96 \times 800}{38\,500} = 2{\cdot}0 \text{ tons per in. run.}$$

Using a fillet on each side of the web, each fillet will require to carry a load of 1·0 ton per in. A $\frac{3}{8}$ in. fillet will be used for the full length of the $\frac{1}{2}$ in. web plate both for tension and compression flanges. At the right-hand end the maximum vertical shear is 66 tons and

$$S = \frac{66 \times 432{\cdot}5}{21\,000} = 1{\cdot}36 \text{ tons per in.}$$

Using two fillets, each will require to carry a load of 0·7 ton per in. A continuous $\frac{1}{4}$ in. fillet will be used on each side of the web.

Stiffeners. The stiffeners over the support at the left-hand end of the girder will carry a shear of 96 tons, which must be transmitted to the web by the welds. If four stiffeners are used, each will carry a load of $96/4 = 24$ tons. Using 6 in. $\times$ $\frac{5}{8}$ in. stiffeners, the compressive stress would be

$$\frac{24}{6 \times \frac{5}{8}} = 6{\cdot}4 \text{ tons per in.}^2,$$

which is reasonable. The load to be transferred to the web by the welds is 24 tons per stiffener. The amount of $\frac{1}{4}$ in. fillet required at 0·9 tons per in. is $24/0{\cdot}9 = 26{\cdot}7$. (Weld 4 in. and miss 6 in. each side of stiffener gives more than sufficient strength.) The stiffeners at each support and under the point load would be made similar for convenience.

Intermediate Stiffeners. Spacing the intermediate stiffeners at 3 ft., the load on each derived from the formula above is 37 tons. These stiffeners may be made 6 in. $\times$ $\frac{1}{2}$ in., which give a stress of 6·2 tons per in.² The amount of $\frac{1}{4}$ in. fillet required is $18{\cdot}5/0{\cdot}9 = 20{\cdot}5$ in. per stiffener. By welding 3 in. on each side of the stiffener at top and bottom, and 3 in. miss 9 on each side, staggered in between, sufficient welding is provided. The stiffeners over the supports are fitted at top and bottom. Those under the point load are fitted at the top only. The stiffeners are welded right round at the top flange, but only for a short length at the outer edge on the bottom flange. The stiffeners have the inner corners cut away to allow the flange-to-web fillet weld to be made continuous.

The butt welds in the flanges and web should be made before the flanges are attached to the web. The stiffeners are usually welded to the web first and help to keep the flanges square.

Struts and Stanchions. For stanchions the use of welding permits the adoption of economical arrangements of the steel which are not possible in riveted construction. For light sections suitable for roof trusses and lattice girders, a number of unusual sections may be used. A box section composed of two angles, Fig. 62 (*a*), or two channels, Fig. 62 (*b*), connected by the toes gives the same radius of gyration about both major axes, and is very economical. Starred angles can

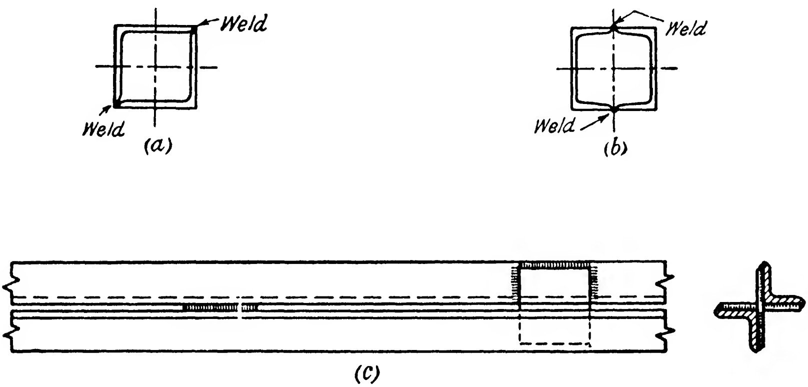


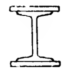
FIG. 62. TYPICAL STRUT SECTIONS

frequently be used to advantage (Fig. 62 (*c*)). Small battens as shown are sufficient to make the two angles act together as one unit.

For steel frame buildings, a section consisting of two rolled steel joists or two channels joined by the toes of the flanges makes an economical substitute for the plated-joist stanchion which is generally used in riveted construction. A comparison of the properties of a stanchion consisting of two 12 in. × 5 in. × 32 lb. R.S.J.'s with a plated joist of similar carrying capacity consisting of 12 in. × 5 in. × 32 lb. R.S.J. with two 9 in. × $\frac{9}{16}$ in. plates in Table VII indicates a substantial margin of saving in the double-joist stanchion.

TABLE VII

COMPARISON OF DOUBLE-JOIST AND PLATED-JOIST STANCHIONS TO CARRY A LOAD OF 84 TONS ON A STANCHION LENGTH OF 14 FT.

Stanchion Section	 2–12 in. × 5 in. × 32 lb. R.S.J.'s	 12 in. × 5 in. × 32 lb. R.S.J. 2–10 in. × $\frac{9}{16}$ in. plates
Weight per ft. (lb.) .	64	72·7
Area (in.^{2}) . . .	18·9	20·75
Length (ft.) . . .	14	14
Radius of gyration (in.) .	2·70	2·24
Ratio l/r . . .	62	75
Working stress by L.C.C. formula for fixed ends (tons per in.^{2}) . .	5·79	5·15
Allowable load (tons) .	109·4	106·8
Number of lines of weld	2	—
Number of lines of rivets	—	4

For greater loads, the arrangement shown in Fig. 63, in which the plates are welded to the toes of the stanchion, may be useful. This section gives a good radius of gyration in both directions and is simple to fabricate. Stanchions consisting of two joists spaced sufficiently far apart to allow the beams to run between them are convenient when continuity in the beams is required (Fig. 64). The two joists may be latticed but are usually connected by batten plates.

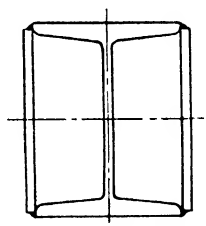
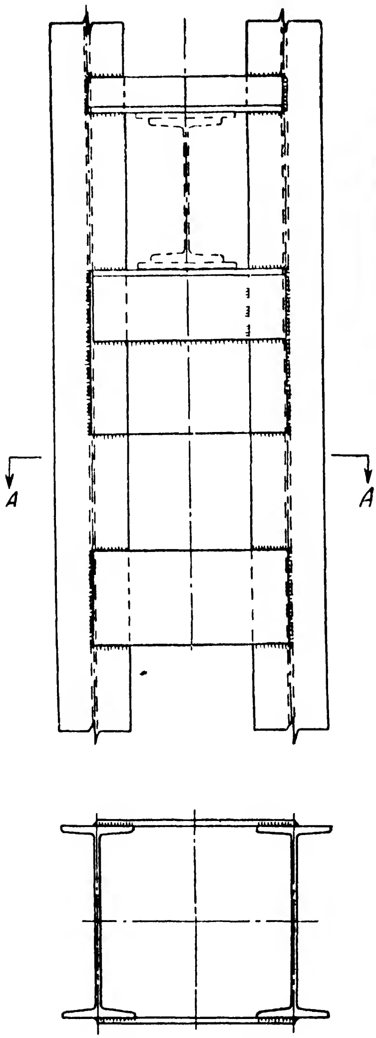


FIG. 63. ALTERNATIVE METHOD OF ATTACHING FLANGE PLATES TO WIDE-FLANGE JOISTS

When heavy loads are to be carried, considerable simplification and saving in labour can be achieved by making the stanchions from flat plates instead of a multiplicity of sections and thin plates. Fig. 66 gives a comparison between a heavy stanchion built up from channels and plates with a welded shaft built from three heavy plates. The welds are proportioned to carry the shear between the parts joined, and are usually quite small.



Section at A-A

Fig. 64. Double Joist Column

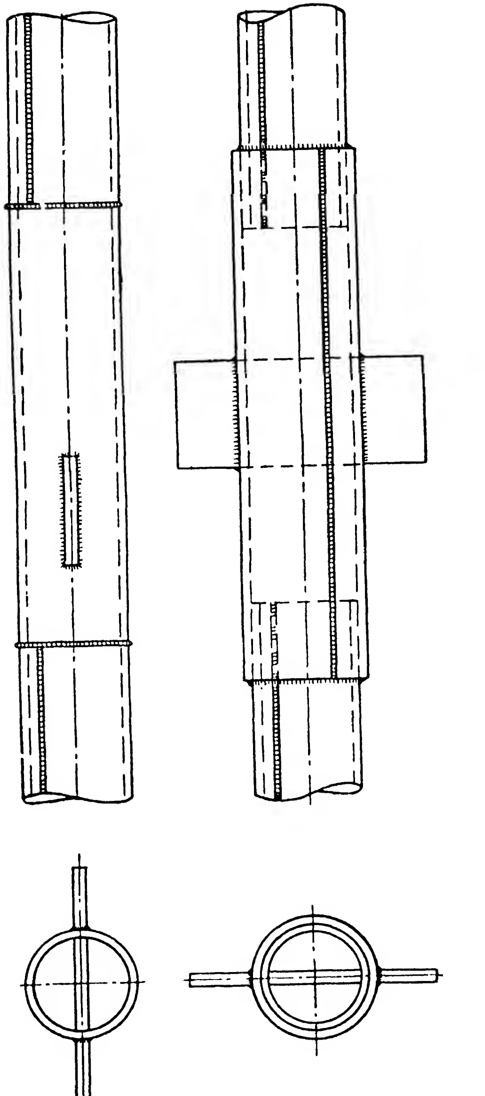


Fig. 65. Tubes as Columns

The lightest possible stanchion section is the cylindrical tube, and, as with welding there is no difficulty in making satisfactory joints, it has many possible applications. The tubes may be joined with butt or lap welds as indicated in Fig. 65.

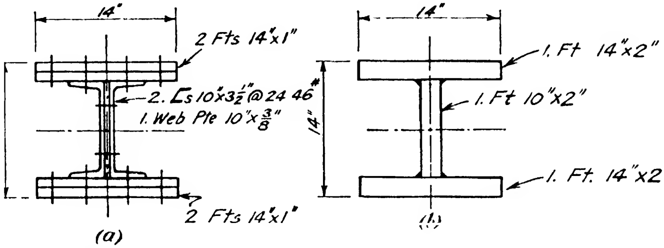


FIG. 66. COMPARATIVE SECTIONS OF STANCHIONS
(*a*) Built up from channels and plates and riveted.
(*b*) Built up of three heavy plates and welded.

Flat plate gussets let through the tube on the centre line are effective. For small sizes the price of the tubes is great in comparison with rolled sections and they are not usually economical on that account, but if tubes are available at reasonable cost they are very effective.

CHAPTER VII

MULTIPLE-STORY STEEL FRAME STRUCTURES

THE use of welding in the fabrication of multiple-story steel frame structures must be discussed in relation to two distinct methods of construction. In ordinary building practice the floor beams are seated on cleats attached to the stanchions and are designed as freely supported beams. In substituting welding for riveting in the attachment of the supporting brackets, the method of calculation of the members of the frame is not affected, and the design of the connections themselves constitutes the only problem.

One of the principal advantages of welding is that it permits the construction of entirely rigid joints and enables the welded frame to be designed as a monolithic structure, taking advantage of the continuity of the beams and stanchions to affect the method of carrying the loads. In general, by designing a structure with rigid connections, the weight of steel in the beams is reduced considerably, but the saving is offset to some extent by an increase in the weight of the stanchions. In a single span portal frame the effect of making the joints rigid usually increases the total weight of the steel, but in the multiple-story structures of several spans the saving may be as much as 20 per cent, provided the steel is arranged in continuous lines symmetrical about the stanchions.

It is important to appreciate an essential difference between welded and riveted connections. The ordinary beam to stanchion connections of riveted construction are all more or less flexible and permit some rotation of the ends of the beams. Furthermore, if a single rivet of a group is overstressed, the rivet material is sufficiently ductile to permit a substantial deformation to take place without fracture occurring. Thus, if the common practice of designing structures as pin jointed and

then making them with semi-rigid joints is followed, no serious danger is introduced so far as the joints are concerned, whatever may be the effect on the stanchions.

With welded connections it is different. The nature of the welded joint is such that almost complete rigidity is induced as soon as two pieces are welded together, up to the limit of the strength of the welding, and consequently it is essential that the joints be arranged to permit freedom of movement, or else designed as completely rigid. If rigid connections are used, the joint must be sufficiently strong to develop the strength of the smaller of the two members joined without overstressing the welds. If the beams are designed as freely supported, the joint must be so arranged that the beams may be able to deflect under load without affecting the magnitude or distribution of the stress on the welds which carry the load.

Structures with Beams Freely Supported. In structures in which the beams are freely supported, the general calculations are the same as for riveted construction.

Beam Brackets. The beams are preferably carried on seating brackets which may be made in a number of forms as shown in Fig. 67. The best form is the simple angle bracket which gives the least eccentricity of load on the stanchion and leaves the beam free to rotate without affecting the stresses on the welds. Some instructive experiments described in the *Journal of the American Welding Society*, March, 1931, show the behaviour of angle brackets with different arrangements of the welds. From the aspect of stress distribution, the most effective arrangement is when the angle may be welded right round with a combination of end and side welds. It is shown that when top and bottom welds only are used, they do not share the load evenly, the top weld taking the major part.

With side welds only, in addition to the fact that they carry a smaller load per linear inch, and are consequently less economical than end welds, there is a likelihood of introducing secondary stresses due to flexure in the plane of the vertical leg of the bracket between the welds, particularly if the bracket

is wide, and there is also some tendency to tearing action at the top end of the welds due to the bending of the horizontal flange of the angle under load. If a weld is made across the top of the angle, the beam must be cut to clear the weld, and this increases the eccentricity of the load and increases the bending stress in the angle, or else the heel of the angle may be chamfered (Fig. 68). The top weld is usually omitted and the weld is made on the sides and the bottom only. The weld across the bottom of the angle obviates any bending in the angle in

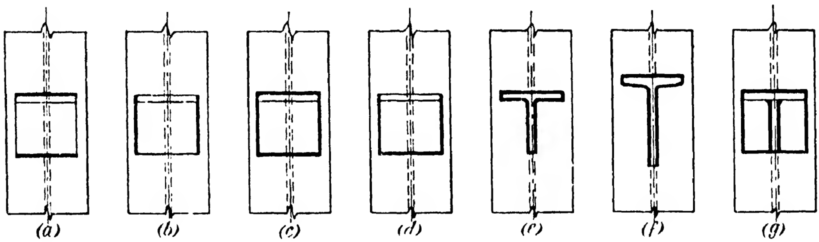


FIG. 67. BEAM SEATING BRACKETS

the plane of the vertical leg, but does not reduce the concentration of stress at the top end of the welds. The tee brackets of Fig. 67 (*e*) and (*f*) are effective, but, in common with all stiffened brackets, they increase the amount of eccentricity.*, †

When a top cleat is used it is welded solidly to the beam but welded to the stanchion along the edge of the toe only, leaving the angle free to open when the beam deflects. If the weld is carried down the sides of the angle, the joint is made rigid and a heavy tearing stress is applied to the ends of the side welds. With the welds arranged as in Fig. 69, it is possible, by making the top cleat of thick material, to apply a partial fixing moment to the beam.

It is usually unsatisfactory to attach a beam to a stanchion

* J. H. Edwards, H. L. Whitemore, and A. H. Stang: "Strength of Welded Shelf Angle Connections." *Bureau of Standards Journal of Research*, 1930, Vol. 5, No. 4. 781–792.

† Tables of safe loads on various types of bracket are given in the *Handbook for Welded Structural Steelwork* (Institute of Welding, 1938).

by a web weld only (Fig. 70 (*a*)), because the weld applies restraint and a fixing moment is developed up to the limit of the strength of the weld as load is applied, and heavy stress occurs at the end of the welds tending to cause tearing in the plate or progressive failure in the welds. If the welds were made of greater cross-sectional area than the plate, any plastic movement which occurred under load might be expected to take place in the plate, leaving the welds intact, but this can hardly be considered to be desirable practice. Since there is usually some clearance between the end of the beam and the stanchion face (Fig. 70 (*b*)), there is also uncertainty as to the effective size and strength of the weld. This difficulty may be overcome by adding plates to the web extending to the stanchion face, but this does not improve the stress distribution in the weld and the joint requires twice the length of weld (Fig. 70 (*c*)).

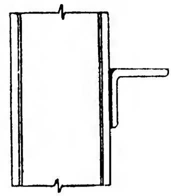


FIG. 68. ANGLE BRACKETS

Angle cleats welded solidly to the web, and attached to the stanchion by fillet welds at the outer edge of the angle only, give a more satisfactory arrangement, because any small rotation is taken up by the angle opening out, and though there may be some inequality of stress along the fillet weld the joint gives sufficient flexibility for adjustments to take place. If erection bolts are used they may be used to relieve the end of the weld of any possible prying action as in Fig. 70 (*e*), and the joint made in this way is quite effective.

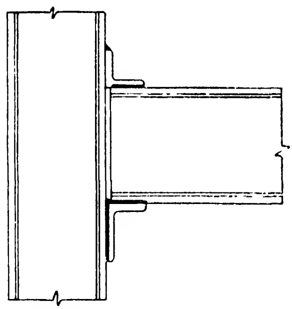


FIG. 69. ATTACHMENT OF TOP ANGLE CLEAT

STANCHION BASES AND CAPS. A flat plate welded directly to the stanchion shaft gives the cheapest and simplest form of base for small loads (Fig. 71). If there is no bending stress to be carried and the loading is direct, the plate is attached to the

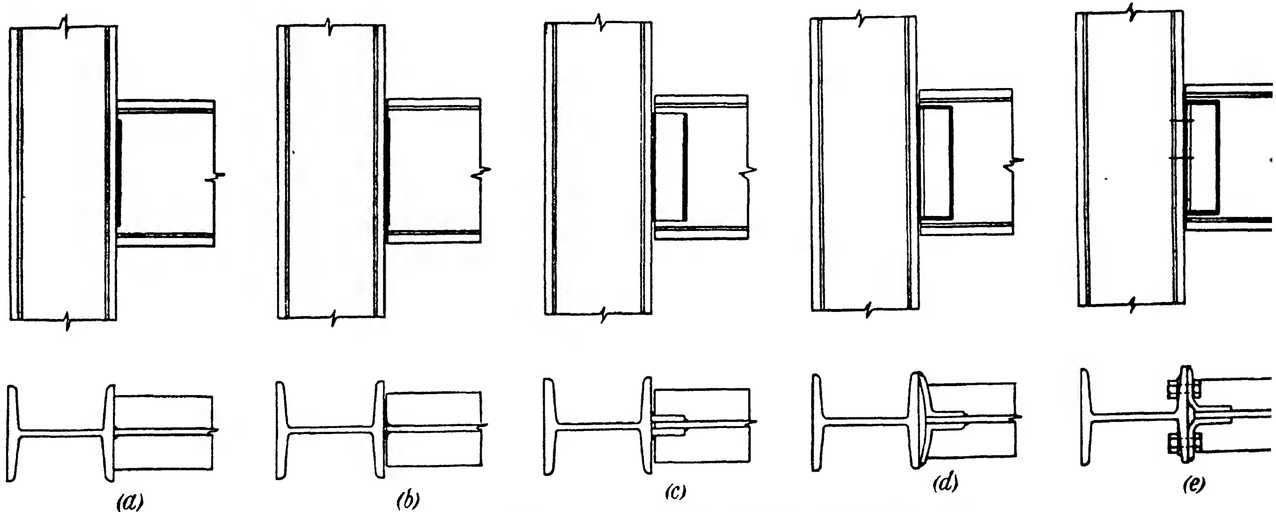


FIG. 70. ATTACHMENT OF BEAMS TO STANCHIONS BY MEANS OF THE WEB ONLY

machined end of the stanchion shaft with small fillet welds which need only be sufficient to ensure safety in transit and fixing. The stress in fixing may be considerable if the stanchion is worked into position by means of a bar under the edge of the base plate. When the end of the stanchion is not machined, and when the heavy bending moments are to be transferred to the base, the welds must be designed to carry the applied loads. When it is necessary to use a built-up base, the base plate may be stiffened by gussets of flat plate welded to the plate and to the stanchion shaft (Fig. 72). Angles are seldom used, and, as far as possible, the load is transferred from comparatively small areas of base plate to the shaft by gussets acting in direct compression, as opposed to the angles of the riveted base which carry heavy bending stresses. For broad flange beam stanchions it is frequently advantageous to attach the gussets to the toes of the flanges as in Fig. 72 (*b*).

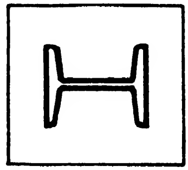


Fig. 71
Flat Plate Base for Small Column Sections

The design of bases is straightforward and requires little

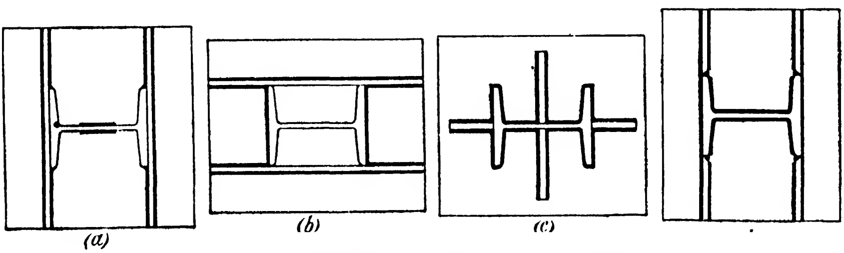


Fig. 72. Built-up Bases for Medium or Heavy Loads

comment. When triangular gussets are used they should be placed in the plane of the flanges or web of the stanchion shaft, or be balanced in pairs in such a way as to avoid introducing bending or torsional stresses into any part of the shaft which may deform laterally under the load (Fig. 72 (*c*) and (*d*)). The base of Fig. 73, in which the gusset plates radiate

diagonally from the toes of the flanges, is defective in this respect, because under comparatively small load the flanges commence to bend inwards at the top of the gussets.

Stanchion caps usually consist of a flat plate welded to the stanchion shaft (Fig. 74). The cap plate may be made to extend just sufficiently beyond the shaft to allow a fillet weld to be made, or it may extend several inches if a wide bearing is required. The cap plate is holed to locate the beams which it carries, and, if necessary, triangular gussets are used to stiffen the plate locally.

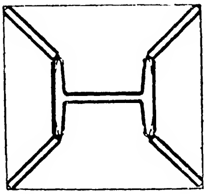


Fig. 73
Base with Gussets Out of Line with Flanges leading to Local Deformation of Flanges

Stanchion Splices. The characteristic welded stanchion splice is made by letting in a bearing plate between the two lengths of shaft (Fig. 75 (*a*)). The plate is welded to the end of the lower shaft in the shop or on the ground, and the connection of the upper length is made in position. Only sufficient welding need be applied to hold the shaft in position, but in practice a weld is usually made right round the shaft. If the shaft is not machined to give a true bearing, the welds must be made sufficiently large to carry the whole load. When

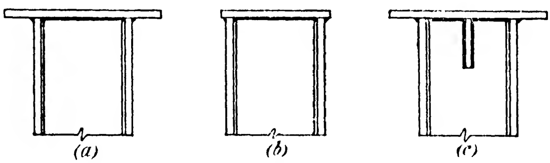


Fig. 74. Stanchion Cap Plate

the stanchion may be subject to heavy bending stresses the weld should be proportioned to carry the full bending moment. If the size of the fillet weld to give the required strength is more than $\frac{1}{2}$ in., it may be economical to chamfer the edges of the stanchion shaft as in Fig. 75 (*b*). A cleat may be welded to the web of the upper shaft holed to match up with the

holes in the division plate. By this arrangement no drilling is done in the stanchion shafts, the holes being all in the small pieces. Since the detail is compact, the splice may be made immediately above floor level. If the size of the stanchion shaft is varied at the splice, the bearing plate must be sufficiently thick to resist the bending stresses introduced. If the variation in size is large, stiffening gussets may be used as in Fig. 75 (*c*).

The splice may also be made with splice plates as in riveted

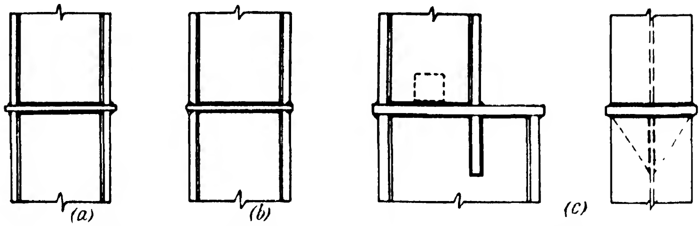


FIG. 75. TYPICAL WELDED STANCHION SPLICES

work (Fig. 76). If the butting ends of the stanchion are machined to ensure true bearing, the welding need only be sufficient to hold the pieces in position. If the ends are not machined, the splice plates and the welds must be designed to carry the whole load. When there are heavy bending moments on the stanchion which may induce tension in the splice, the plates and welds should be strong enough to carry the bending. In order to leave room for applying the welds, the flange plates are made either wider or narrower than the flanges of the stanchions (Fig. 76 (*a*) and (*b*)). Erection holes may be made in the splice plates for locating the upper shaft. When the upper shaft is smaller than the lower, packers must be introduced, and these necessitate additional lines of weld, which add substantially to the cost (Fig. 76 (*c*)). Plated splices of this type are usually made far enough above the floor level to keep the splice separate from the connection details. If the splice plates are attached to the lower shaft in the shop or on the ground, the welds are stopped two or three

inches from the end of the shaft to enable the plates to be sprung open to take the upper shaft. If the welds extend to the end of the shaft, the shrinkage of the weld may tend to deflect the ends of the plates inwards, making it difficult to let in the upper shaft.

Structures with Rigid Joints. In general, whenever members of a steel frame are connected directly by means of butt welds or fillet welds, the strength of the welded joint must be such as

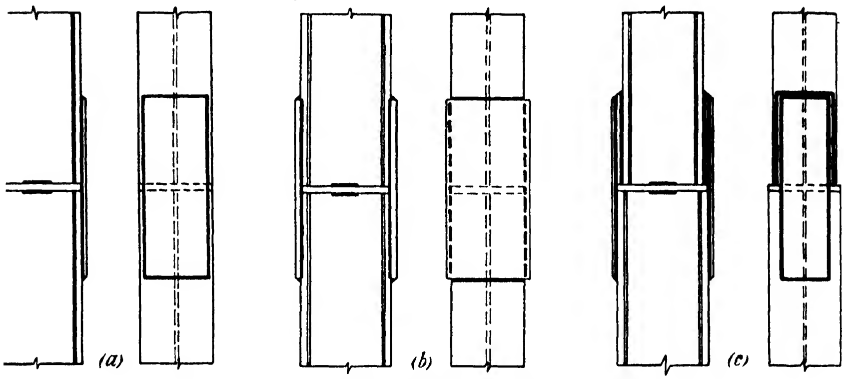


Fig. 76. Stanchion Splices with Cover Plates

to develop the full effective strength of the members, and the structure must be designed as a rigid frame. The fact that a frame is statically sound when the beams are freely supported is no guarantee that they will not be overstressed if the joints are made rigid. The stresses induced in a stanchion by a beam connected to it rigidly may be sufficient to overstress it, though it could readily carry the vertical load if applied centrally or with normal eccentricities. On the other hand, by designing a structure as a rigid frame, the weight of steel can usually be substantially reduced.

It is beyond the scope of this book to discuss the design of rigid frame structures, but the effect of using rigid joints on the distribution of stress will serve to indicate the advantages and limitations of this form of construction.

For a beam simply supported at the ends and carrying a uniformly distributed load, the maximum bending moment occurs at the centre of the beam and has the value $M = Wl/8$. If the ends of the beam are completely fixed, a maximum bending moment of value $M = Wl/12$ occurs at each end of the beam, and the moment at the centre of the beam is $M = Wl/24$; that is to say, the maximum moment in the fixed beam which occurs at the end of the beam is two-thirds of the maximum moment in the simply supported beam, while the moment at the centre of the fixed beam is only one-third of the corresponding moment in the simply supported beam. It is therefore apparent that, in so far as the beams of a structure are concerned, a substantial reduction in the bending moments results from fixing the ends.

When the ends of a beam are fixed rigidly to two supporting stanchions making a simple portal frame, rotation takes place at the joints when the beam is loaded, depending on the relative stiffness of the beam and stanchion, and the distribution of the bending moments is modified accordingly. When the stiffness of the beam and the stanchion are similar, the maximum bending moment at the centre of the beam has the value $M = Wl/14{\cdot}4$, while the bending moment at the ends has the value $M = Wl/18$.

When the beam is connected rigidly to the stanchions there is thus a substantial reduction in the maximum bending moment in the beam and a corresponding increase in the bending moment in the stanchions. When the length of the beam is short in comparison with the length of the stanchions, this redistribution of moments may lead to an increase in the total weight of the frame, but if the length of the beam is great in comparison with that of the stanchions, there may be a total overall saving. The advantage of using a single frame portal in a building with wide openings is that it enables the depth of the beams, and consequently the height of the building, to be reduced without appreciable alteration in the weight of the steelwork. At the same time the portal frame provides

resistance against wind and lateral loads. When a frame is made up of a number of connected portals, the advantage of fixed connections is increased, as is indicated by the diagrams of Fig. 77, which give the shape of the bending moment diagrams for the beams and stanchions for a single-story and two-story portal frame. It is seen that when the stanchion continues above the beam, the moment introduced into the stanchion is shared between the parts above and below. If the beams are

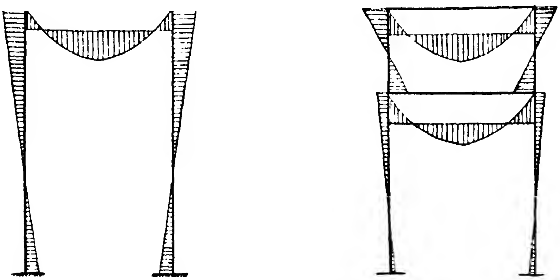


FIG. 77. SHAPE OF BENDING MOMENT DIAGRAM FOR SINGLE AND DOUBLE PORTAL FRAMES UNDER UNIFORM VERTICAL LOADING

continuous over several spans, large bending moments are introduced into the outside stanchions only, the negative moments in the beams being balanced between themselves, and the interior stanchions are almost unaffected when the loading is uniform. Under these circumstances a substantial reduction in weight results from making the joints rigid.

Examination of the bending moment diagrams for the stanchions shows that the maximum bending moments occur near the joints. In designing stanchions by the usual formulae, the allowable working stress is determined in relation to the section at the middle height of the stanchion. For the combined compression and bending stress near the rigid joint there is no reason why the working stress should not be the same as for the beams.

CONNECTION DETAILS. The details of the rigid joint connection of beams to stanchions may be made in a variety of

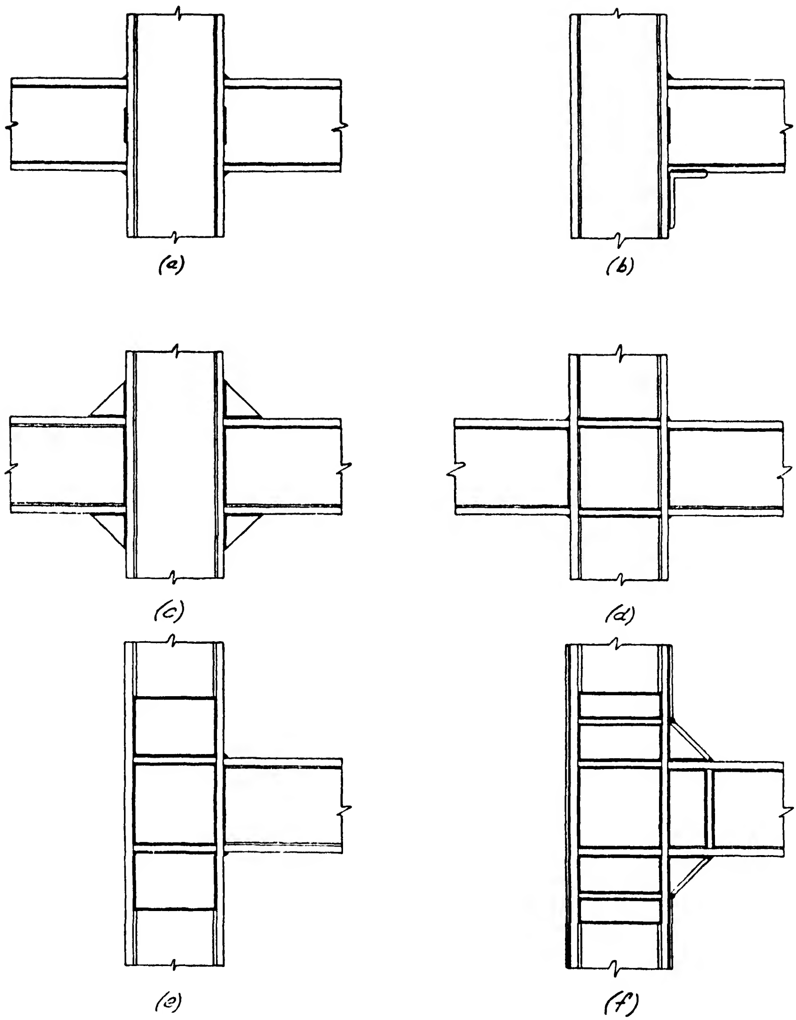


Fig. 78. Rigid Beam-to-column Connections

ways. Fig. 78 gives several details for a joint between simple joists. In Fig. 78 (*a*) the joists are attached by fillet welds round the periphery of the beam. In Fig. 78 (*b*) a landing bracket is used to facilitate erection. On the compression side of the beam the bracket does not introduce any serious difficulties, but a bracket on the tension side involves the problem of avoiding concentration of stress at the ends of the welds. In all cases where heavy bending moments are to be transmitted through the joint, care must be taken that the distribution of stress is not affected by the relative flexibility of any of the parts. This is well illustrated by considering the method of failure of a connection of type (*a*) under a crippling load. The first failure in such a joint is by the buckling of the stanchion web under the compressive reaction from the beam. If the stanchion web is supported by stiffeners to obviate this overstressing, failure will usually occur by the stretching of the stanchion web under the tension reaction from the beam. If the weld joining the beam to the flange of the stanchion is relatively light, it is possible that failure may occur in this weld owing to the unsupported flange of the stanchion deforming as in Fig. 39 (*a*), and causing a concentration of stress at the middle part of the weld and progressive failure of the weld from the middle. If additional stiffeners are introduced to support the tension load, failure will occur by buckling in the web of the beam at some distance from the joint. The behaviour of such a joint indicates that there is no difficulty in making the welds sufficiently strong to develop the strength of the members, but that careful attention must be given to the distribution of stress throughout the joint. It is clear that the joint of type (*a*) is only suitable when the bending moments to be transferred are quite small. For moderate loads the reaction from the beams may be spread along the web of the stanchion by triangular gussets in the plane of the webs as in Fig. 78 (*c*), but when the bending moments are heavy it is preferable to use stiffeners in continuation of the flanges of the beam as in Fig. 78 (*d*). When large moments are to be transferred from the beam to the stanchion, heavy shear

stresses occur in the web of the stanchion, and it may be necessary to reinforce it with additional web plates as in Fig. 78 (*e*).

The arrangement of rigid joints for plated stanchions is rather difficult. It is necessary that the beam be connected directly to the stanchion shaft, because if it were attached to the plate it would merely pull it off. An example of the connection of a beam to a plated stanchion is shown in Fig. 78 (*f*). For rigid frame construction, stanchion shafts built from three flats will be preferred to plated stanchions.

A double-joist stanchion, with the joists spaced sufficiently far apart for the girders to be threaded between them, has been used widely for high buildings and, while it has the obvious disadvantage of taking up more room than a single joist plated, or two joists welded toe to toe, it has several useful features. If the beams are stopped at the stanchion, the full negative moment must be transferred through the welded joints, whereas, if the beams can be passed through the stanchion, the joint in the beam is avoided altogether or made at a place where the bending moment is a minimum. When, for the sake of economy, it is desirable to proportion the beam section to carry the bending moment at the centre of the beam and to reinforce it near the supports to carry the greater negative moment by the addition of extra plates to the flanges or by the provision of haunches, this is greatly facilitated by the use of the double-joist stanchion. Since the bending stresses in the stanchions are greatest near the joints, brackets supporting the beams may be arranged to give the additional strength locally, thus allowing the weight of the shaft to be kept down. A detail involving the use of double-joist stanchions was given in Fig. 64.

CHAPTER VIII

ROOF FRAMES: TRUSSES

Roof Frames. The unattractiveness of the conventional roof truss and the difficulty of fitting it into any scheme of architectural treatment is leading to the use of portal frames built of rolled or fabricated sections to replace the more usual stanchion and roof truss arrangement. The tendency has been delayed by the fact that in riveted construction these frames are expensive to fabricate, but with the advent of welding their use is increasing rapidly.

The arrangement of the frame varies according to the type and shape of the building. The frame may be made rigid or with two or three hinges. The three-hinge frame being statically determinate is the simplest form and is frequently preferred on account of the ease with which the stresses may be calculated. The bending moment varies from a maximum at the corners to zero at the apex and at the points of support. In Fig. 79, rolled steel joists are used for the stanchions and rafters and the corner is stiffened by a joist cutting. In Fig. 80, where the span is much greater, the depth of the sections varies from a minimum near the hinges to a maximum at the corners, thus conforming generally to the shape of the bending moment diagram. The frames are built up of flat plates, the webs being stiffened by flat stiffeners. Special attention has been paid to the stiffening of the corner, and the inner flange which carries compressive stress at this point has been made of channel form by welding flat plates to the edges of the main plates.

A two-hinge frame involves more calculation but is usually lighter than the three-hinge frame. There are points of maximum bending moment at the corners and also at the middle of the span. In Fig. 81 the shape of the two-hinge frame conforms to the shape of the bending moment diagram, being deep at the

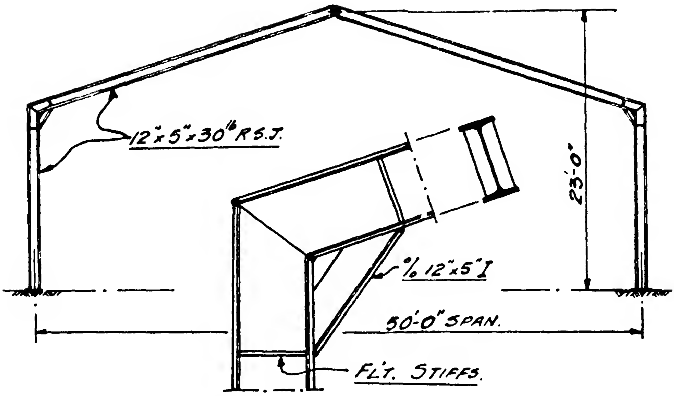


Fig. 79. Three-hinge Portal Frame with R.S.J. Rafters and Columns

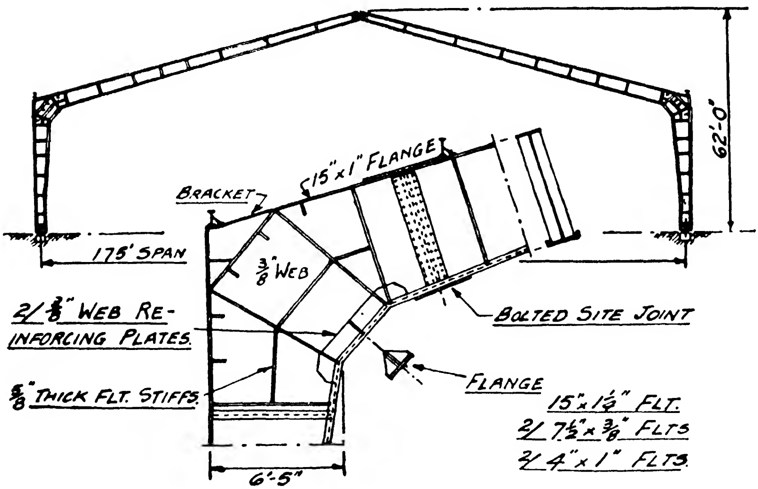


Fig. 80. Three-hinge Portal Frame of Wide Span with Frames Built from Plates

corners and at the middle of the span and less deep at the points of no moment.

When it is possible to restrain the frame at the supports, a continuous rib without hinges is usually most economical. The calculations, which are made either by a slope deflection method

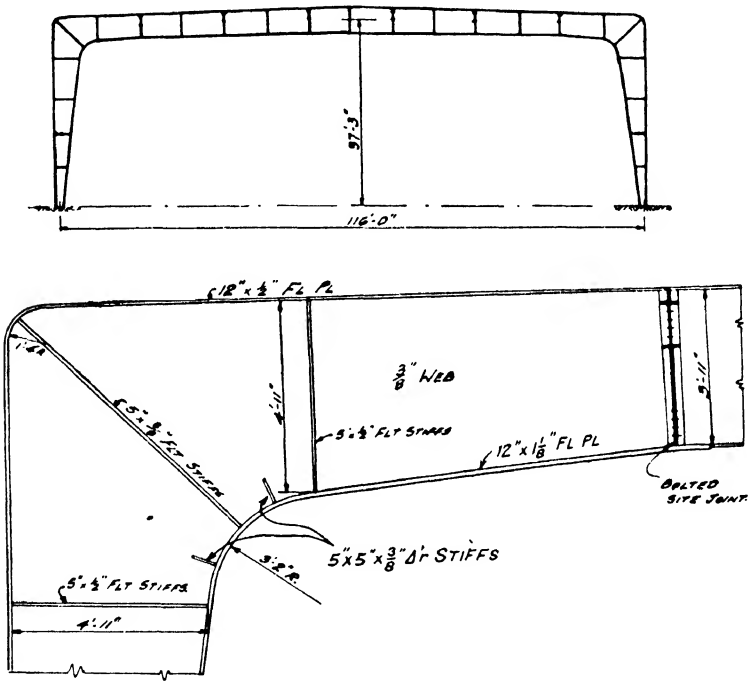


FIG. 81. TWO-HINGE PORTAL FRAME OF 116-FT. SPAN

or the principle of virtual work, are rather laborious. The stresses in the frames may, however, be analysed readily by mechanical methods.

Two examples are illustrated. In Fig. 82 the roof frame springs from the girder supporting the floor and is welded rigidly to it. The shape of the frame follows the line of thrust for uniform loading and the bending moments at all points are small. In Fig. 83 the roof frame is calculated as part of the

general framework by the slope deflection method and the bending moments are distributed between the members of the frame in proportion to their stiffness I/l. The use of the rigid joints results in a considerable reduction in the maximum bending moment in the main beam and the rafters, with an increase in bending moments in the stanchions.

In comparing the simple truss with a roof frame, unless the frame may be made to conform closely with the line of thrust

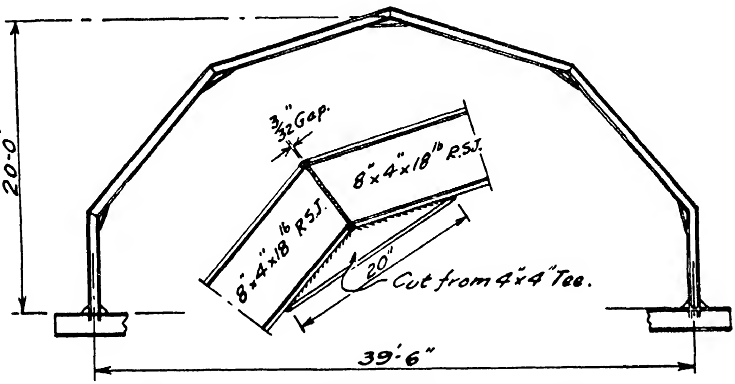


Fig. 82. Roof Frame with Rigid Joints

the roof truss will be the lighter, but the frame offers compensating advantages in that it involves much less obstruction to light and is more pleasing to the eye. When headroom inside the building is an important matter the use of a frame may result in a reduction in the height of the walls and the area of the roof covering. If, in a building with a travelling crane, the crane girder may be shaped to fit within the form of the frame, the overall height of the building may be reduced.

The design of the members and details of frames with solid webs follows the principles used in the design of plate girders. Special care must be taken in analysing the stress conditions at the corners or other places where abrupt change of section or direction takes place. By the use of stiffeners and triangular gussets, the stresses can usually be distributed without difficulty,

but the problems are apt to be complex and to demand experience and good judgment on the part of the designer.

Truss Girders. The design of welded roof trusses as outlined in Chapter VI is very simple. In general the members are in direct tension or compression and the welds can be so arranged that there is little uncertainty as to the distribution of stress in

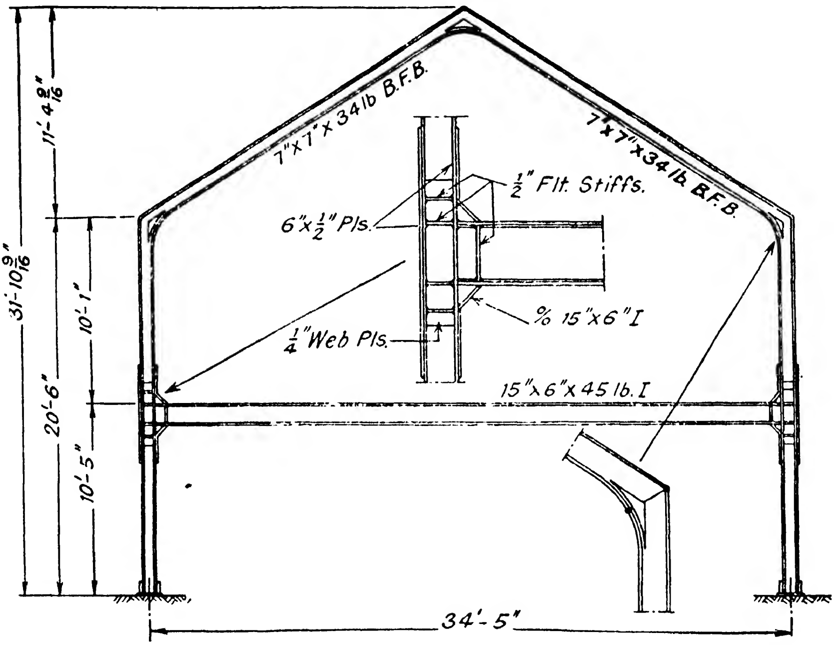


FIG. 83. TWO STORY PORTAL FRAME STRUCTURE

the welds or in the members. When the loads are small there is always a tendency to make the joints stronger than is necessary.

The design of heavy trusses follows the same general principles; but owing to the rigidity of the welded joints, the secondary stresses induced must be taken into consideration in determining the shape of the joints and in proportioning the welds. The substitution of welding for riveting without

altering the arrangement of the joints usually gives unsatisfactory results. Examination of the conditions at the junction of a flat diagonal member in a Pratt-type truss (Fig. 84) connected by welding along the sides instead of rivets through the flat, indicates the problem. The deflection of the truss under load induces a bending moment in the flat and the stress is a

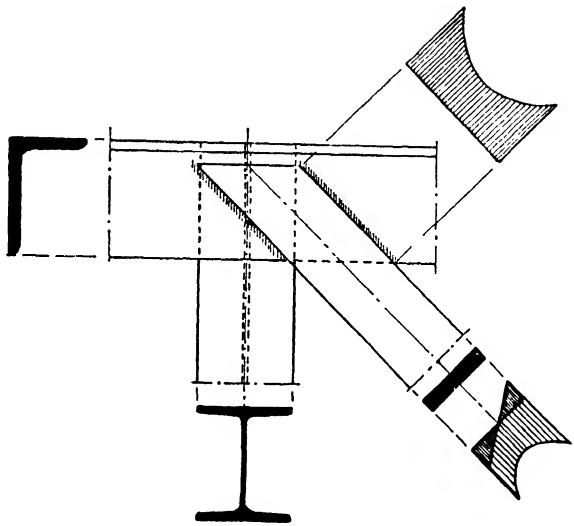


FIG. 84. DETAIL OF CONNECTION OF WEB MEMBERS TO CHORD IN HEAVY TRUSS INDICATING POSSIBILITY OF STRESS CONCENTRATIONS AT THE JOINTS

maximum at the edge of the flat where it joins the gusset. If the flat is connected to the gusset by side fillet welds only, there is also a concentration of stress at the edges of the plate as previously indicated in Fig. 15 (page 23), and at the same time there is also a concentration of stress at the ends of the welds due to the uneven distribution of stress along the side fillet as previously indicated in Fig. 10. Since the concentration of stress from the three separate causes may all occur at one point, a dangerous condition may easily be produced, and it is, therefore, necessary to give very careful consideration to

the design of the joint. In the case of a structure carrying dynamic or alternating stresses, the joint of Fig. 84 is not desirable, and the arrangement of the joint must be modified to obviate the stress concentrations.

The discussion of stress distribution on pages 23 to 26 indicates how the stress conditions in the joint may be ameliorated. The use of a slot weld would render the distribution of stress more uniform across the section of the flat, and reduction in the length of the welds would improve the stress distribution in the welds. The use of end welds in conjunction with the side welds would further improve the stress distribution in the diagonal. Channels might be used as diagonals and the weight of metal in the flanges would then compensate for the concentration of stress along the edges, or the edges of the flat diagonals might be reinforced with edge covers or small gussets to give the same effect. As far as possible the use of gussets as they appear in riveted work is avoided, and forms of joints which do not involve indeterminate stress concentrations are used.

Portal Frame Trusses. The portal frame truss which has no diagonal members, and in which the shear is carried by virtue of the stiffness of the vertical and horizontal members of the truss, lends itself readily to welded construction and is coming to be used largely in some countries when dynamic or alternating loads may be expected, and also on account of its good appearance. In providing completely rigid joints, it avoids the indeterminate secondary stresses of the triangulated truss because the joints are designed to carry bending moments. Extended tests on scale and full-size test models indicate that these trusses are capable of standing up to severe conditions of alternating stress and dynamic loading, and that the joints may be so arranged and the welds proportioned that there is no tendency for failure to occur in the welds. When made with parallel chords, the truss is heavy, for the whole of the shear must be transferred to the chord by the stiffness of the members, through the rigid joints. When made with a curved

top chord, the shear is carried by the top chord and the stresses in the web members are comparatively small. It is in this form that the truss has been applied with success to medium

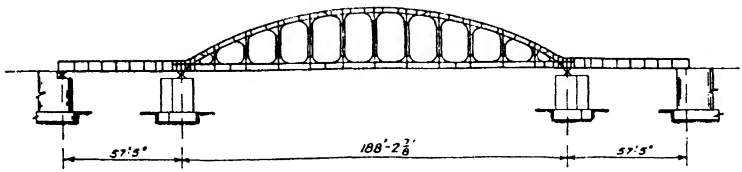


FIG. 85. SKETCH ELEVATION OF BRIDGE WITH PORTAL FRAME GIRDERS

span highway bridges. A sketch elevation of a highway bridge over the Albert Canal at Herenthals, with a span of 188 ft., is given in Fig. 85, and a sketch of the detail at the abutment

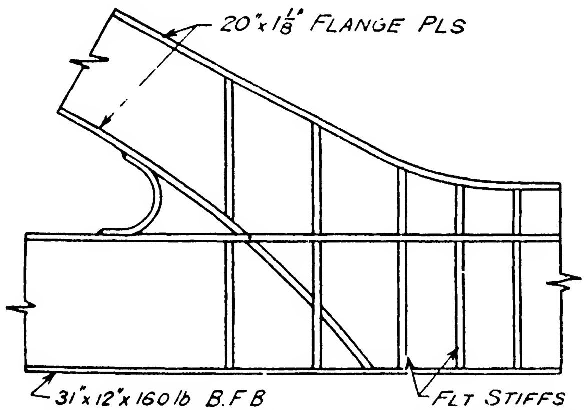


FIG. 86. DETAIL AT ABUTMENT OF PORTAL FRAME GIRDER

is given in Fig. 86, which indicates the ease with which the rather unusual shapes can be built up with flat plates.

CHAPTER IX

WELDING IN REINFORCED CONCRETE CONSTRUCTION

Reinforcements. It is frequently desirable to join reinforcing bars by welding on account of the lack of space for making the usual lapped connection or for other reasons. For diameters up to $\frac{3}{4}$ in., the bars are usually lapped and joined by side fillet welds as in Fig. 87. The welds are proportioned to develop the tensile strength of the bars. The bending moment due to eccentricity is counteracted by the side thrust of the concrete.

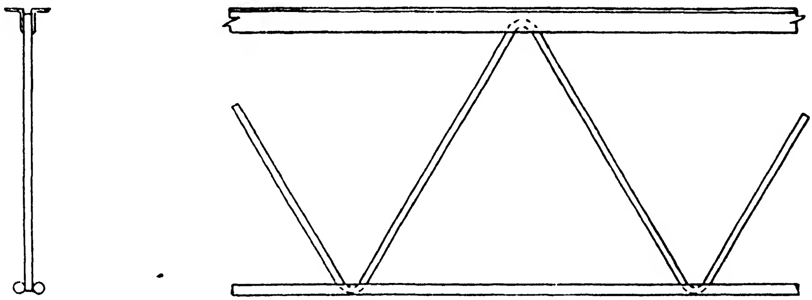


FIG. 89. LIGHT WEIGHT BEAMS FOR COMPOSITE STEEL AND CONCRETE CONSTRUCTION

For bars of larger size a double-V or double-U butt weld is used as in Fig. 88 (*a*) and (*b*). If the welds are made *in situ*, the weld metal is deposited in beads welding from the bottom upwards. Small plates are sometimes welded underneath the bars to give a base from which to commence the weld as in Fig. 88 (*c*), but it is difficult to ensure that slag is not trapped at the bottom of the weld. The plain double-V weld without reinforcing plate is to be preferred.

Composite Steel and Concrete Construction. A method of construction which is likely to be developed widely in the future is the use of concrete and steel in combination. By

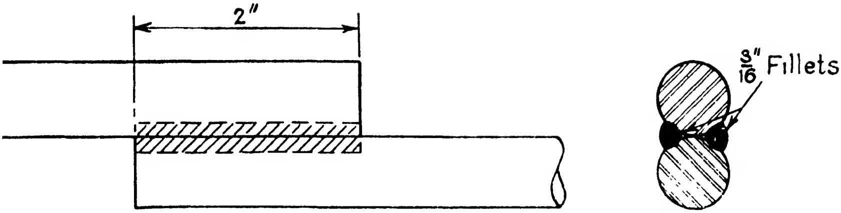


FIG. 87. LAP WELDING OF SMALL SIZE REINFORCING BARS

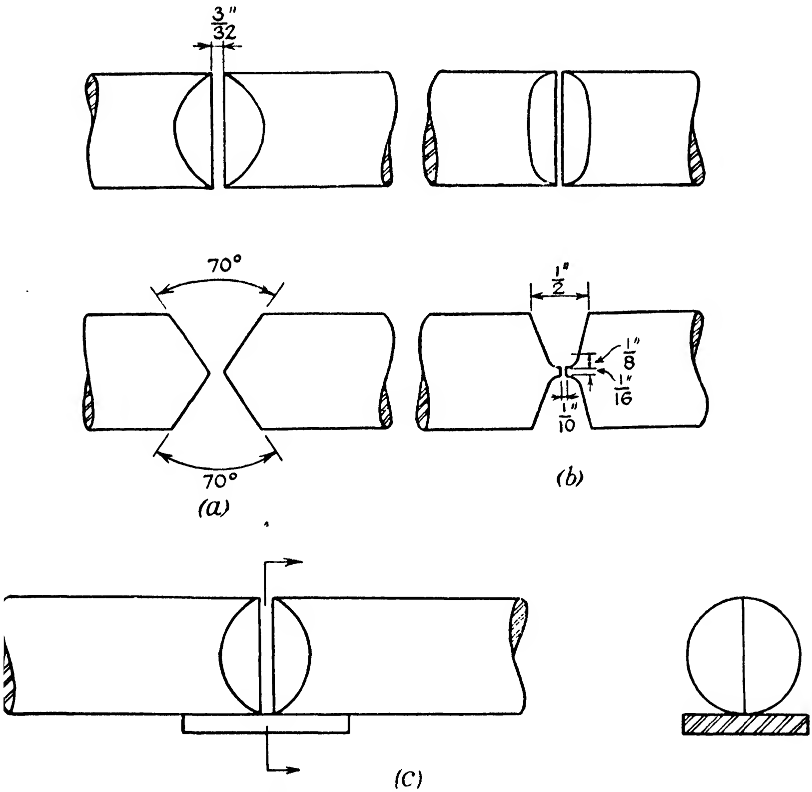


FIG. 88. BUTT WELDING OF REINFORCING BARS

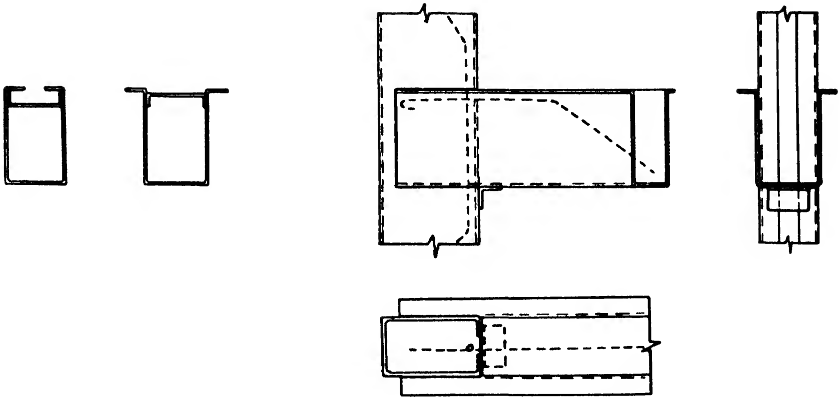


Fig. 90. Hollow Pressed Steel Members for Composite Steel and Concrete Construction

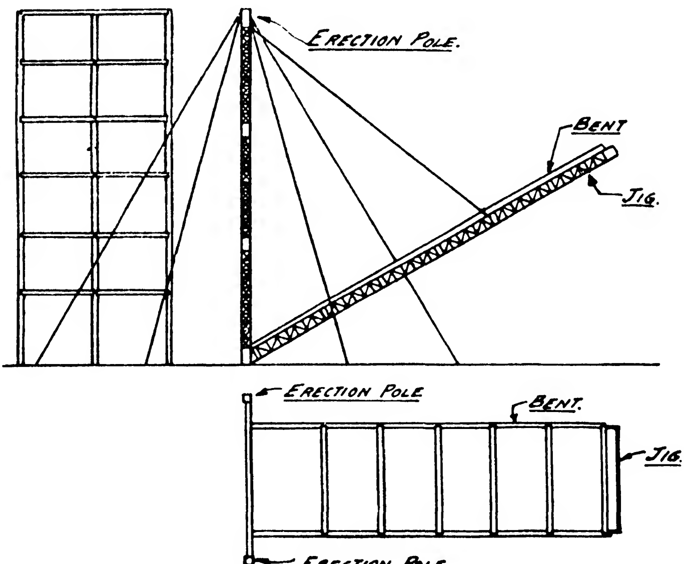


Fig. 91. Erection of Six-story Bent of Steelwork in One Piece

erecting a light steel framework which is sufficiently strong by itself to carry the dead load of the structure, and which afterwards acts as part of the reinforcement of the concrete. the advantages of both methods of construction are combined. The welded joint is well adapted to the requirements of the light framework. A detail of the light steel beams for this type of construction is given in Fig. 89. The web member is formed by a continuous member bent to shape.

An interesting variation of this form of construction is the use of hollow pressed steel sections filled with concrete. One such arrangement is indicated in Fig. 90, which shows the shape of the beam and stanchion sections and the detail of the connections between them. Additional steel in the form of round bars can be set in the concrete at the joints to take the negative moment from the beam. The method of erection of a steel framework of this type used for a block of tenement buildings is of special interest. A complete bent of steelwork for six floors was assembled and welded in a jig on the ground and erected in one piece supported by the jig, as indicated in Fig. 91.

CHAPTER X

FABRICATION

THE method of fabrication of welded steelwork varies widely according to the type of the structure and the facilities available. When a structure may be built of rolled sections which do not require preparation before assembly, the steel may be shipped direct from the mills to the site cut to exact length, without going into the construction shop, and any minor preparation such as the attachment of landing cleats may be done at site. When the amount of welding in the preparation of the members is great, as for girders or stanchions built up from flat plates or when girders are provided with haunches for continuous beam construction, the bulk of the welding is done in the workshop where better handling facilities are available, and the material is taken to the site in the largest pieces possible. The distribution of the work between the shop and the site may be dictated by the conditions at site. Where space is available it may be worth while to fabricate roof trusses at site because they are bulky to transport and take up much room in the shop. Unless handling facilities are available it is preferable to do the welding on heavy pieces in the shop.

In the workshop the method of fabrication depends on the nature of the work. For light work the first requirement is some form of welding table on which the work may be set up and held during assembly. For trusses, a table consisting of a series of joists three to six feet apart set to give a flat level surface two or three feet above the floor, is very useful. The lay-out for such a table prepared for the assembly of a batch of roof trusses is given in Fig. 92. The location of the members is determined by cleats welded to the tops of the joists in appropriate positions. The cleats are designed to hold the members rigidly in position for tacking. On such a table various cleats,

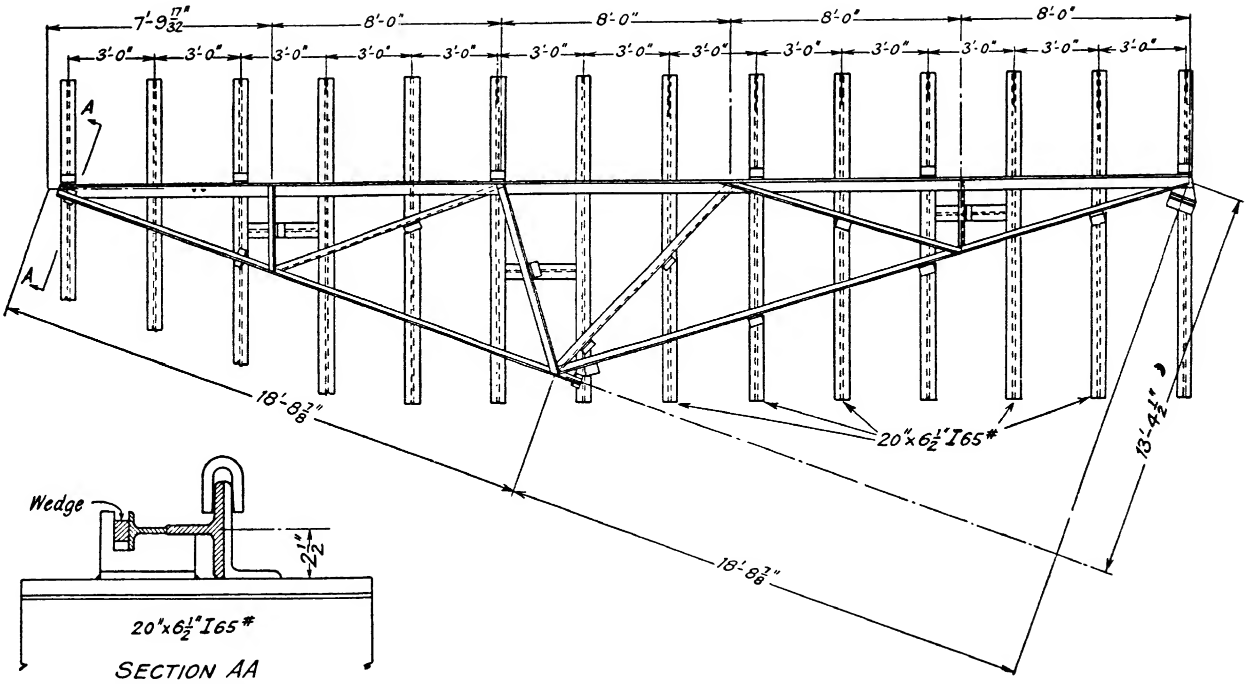


FIG. 92. WELDING TABLE ARRANGED FOR ASSEMBLING TRUSSES

brackets, and packers may be attached for assembling plate girders and other shapes. Level surface tables of cast iron are sometimes used and are very effective though more expensive. It is not possible to weld to the tops of these tables, and efficient holding devices are required.

A useful type of jig for assembling built-up girders of medium size is illustrated in Fig. 93. The plates forming the girder are clamped against adjustable platforms attached to circular frames which rest on rollers and are rotated to turn the girder to the best position for welding.

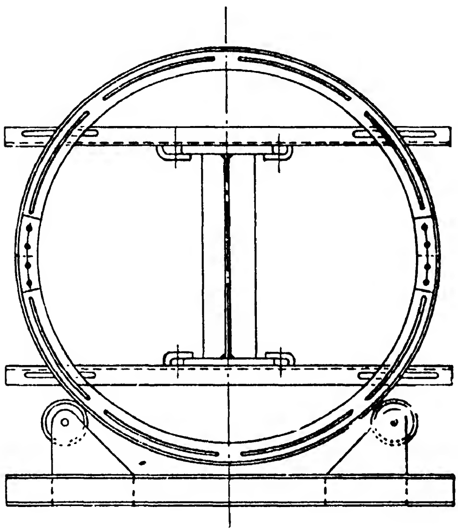


Fig. 93. Rotating Jig for Assembling and Welding Plate Girders or Other Structural Units

Distortion. A great amount of heat is liberated during the welding operation and the stresses induced in the work by the expansion and contraction of the heated metal may be such as to cause distortion in the work. No complete explanation of the phenomenon has yet been offered and the subject is difficult to investigate, but from the practical standpoint the problem of distortion is no longer one of great difficulty in structural work.

If a bar of metal which is prevented from contracting is cooled through 100° C., the stress induced in the metal is of the order of 15 tons per in.^{2} Since the weld metal is deposited in a molten condition at a temperature above 1 600° C., the weld metal in solidifying and cooling must be subjected to stresses which exceed the yield point and cause it to yield.

It is apparent that distortion may be caused by the shrinking of the solidifying weld metal and the subsequent cooling of the

welded joint. The magnitude of the movements and forces involved may be judged from the fact that a small run of metal (e.g. 10/9) round the outside of a 6 in. pipe 0·35 in. thick causes the pipe to shrink about 0·007 in. To cause an extension or reduction of this amount in a piece of steel 10 in. long would require a load of 15 tons per in.^{2}, whereas the length in which this contraction actually takes place is of the order of $\frac{1}{4}$ in.

It is therefore evident that whenever weld metal is deposited, both it and the adjacent metal are stressed beyond the yield point during cooling to such an extent that plastic yield takes place. If the ductility of the weld metal is of the order of that of mild steel, the effect of a small permanent elongation on the properties of the metal is very small. While the elongation of a bar of mild steel 10 in. long before the yield point is reached is of the order of 7/1 000 in., the elongation before eventual fracture may be about $2\frac{1}{2}$ in. It can sustain an elongation of limited amount without damage.

If, on the other hand, the total ultimate elongation of the weld metal is small, the stresses due to cooling may cause the weld to break, and in attempting to make butt welds with bare wire electrodes which give a weld metal of about 3 to 6 per cent elongation, the welds usually break during cooling if the ends are restrained from moving in to balance the shrinkage. With weld metals of high ductility and tensile strength, butt welds under restraint may be made without difficulty.

In the first welded structures the distortion of the work during welding caused a great deal of trouble, but gradually as the nature of the problem has come to be understood the difficulties have been overcome.

An elementary discussion of the effect of shrinkage on the two principal forms of weld—butt and fillet welds—helps to an appreciation of the shrinkage phenomena and the methods of obviating distortion.

Consider first the behaviour of a butt weld made between two

pieces of plate with the edges chamfered to form a V as in Fig. 94 (*a*). Before welding, the two plates are in line and separated by a small gap. When welding is in progress, the weld metal is poured into the vee in a molten condition and in shrinking draws the edges of the vee together. The final form of the weld is as shown in Fig. 94 (*b*). If the left-hand plate is held stationary the right-hand plate is displaced as a whole inwards, and an angular rotation takes place about the edge of the fixed plate.

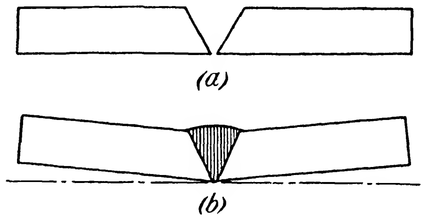


FIG. 94. DISTORTION OF VEE BUTT WELD IN COOLING

In practice, the two plates must be in the same plane after welding. Two methods may be adopted. Either the two plates will be set up out of line by the amount of the distortion and the shrinkage relied on to bring them into line (Fig. 95) or else the two plates will be rigidly held by any method available so that distortion cannot take place, in which case the weld metal stretches to balance the shrinkage and no rotation about the weld takes place. As a rule it is not possible to prevent shrinkage taking place in the plane of the plates and this must be allowed for. This shrinkage is of the order of $\frac{1}{16}$ in. per joint for plates up to $\frac{1}{2}$ in. and small runs of welding.

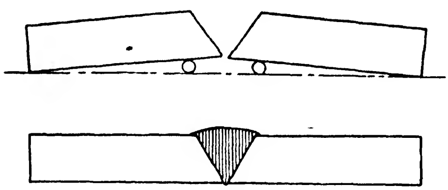


FIG. 95. PLATES SET UP OUT OF LINE FOR WELDING

When the plates are chamfered on both sides to give a double-V as in Fig. 96, if the runs of welding are deposited alternatively on each side the shrinkage on one side balances the shrinkage on the other and the work may be kept straight. If the whole of the welding is done on one side first, distortion will

occur and the welding on the other side will not be able to pull it straight.

In addition to the shrinkage across the seam, there is also shrinkage along the seam, but of much smaller magnitude.

The effect of longitudinal shrinkage is seen more readily in the case of the fillet weld. In a fillet weld 180 ft. long in $\frac{3}{16}$ in. plate a shrinkage of 2 in. was observed in the length of the seam. The weld was made in a single run and the section of the weld was great when compared with the thickness of the plate (Fig. 97 (*a*)). In a fillet weld 120 ft. long in $\frac{3}{8}$ in. plate using three runs no shrinkage occurred. Each run of weld was of the same size

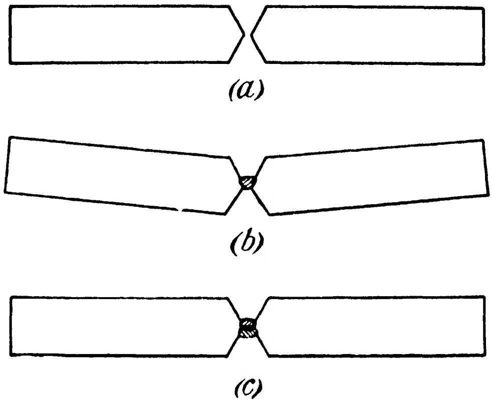


FIG. 96. PULL OF WELD ON ONE SIDE OF VEE COUNTERACTING WELD ON OTHER SIDE

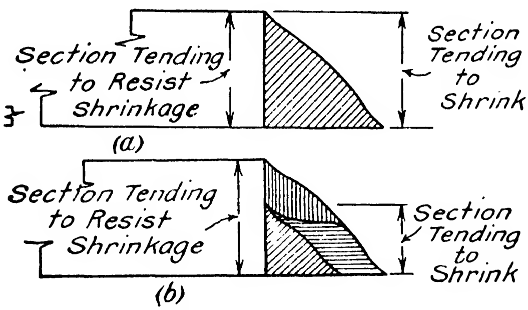


FIG. 97. SHRINKAGE EFFECTS IN FILLET WELDS

as in the previous case, but they were small in comparison with the thickness of the plate (Fig. 97 (*b*)). It therefore appears that if the size of the run of weld is small in comparison with the thickness of the work, little or no distortion

takes place along the weld and the shrinkage is balanced by stretching in the weld metal. If there is no restraint laterally, one plate will be displaced relative to the other by about $\frac{1}{16}$ in. The displacement takes place during the making of the first run only.

There is always a tendency for rotation of the pieces joined by fillet welds. In many cases the form of the pieces joined gives sufficient stiffness to resist the tendency to rotate but where there is no such stiffness the pieces must be restrained. In fabricating a girder from three flat plates (Fig. 98 (*a*)), if the weld joining the web to the flanges were made from one side of the web only, there would be considerable rotation of the flanges as in Fig. 98 (*b*). If the welds are made from both sides of the web in short lengths the shape is preserved, but if the welds are large in comparison to the thickness of the plates, the flange plates tend to fold about the lines of weld (Fig. 98 (*c*)). To avoid this the plate is clamped to a stiff plate or to a pile of plates sufficiently strong to resist the shrinkage force. Alternatively two girders may be set up flange to flange and the welding done on both at the same time so that the pull of the welding on one will counteract the pull on the other (Fig. 99). If there are web stiffeners they are usually welded to the web first and help to keep the flanges true to shape, but, if the web to flange weld is large, the flange plates may be distorted between the stiffeners. If the flange plates are 1 in. or more in thickness,

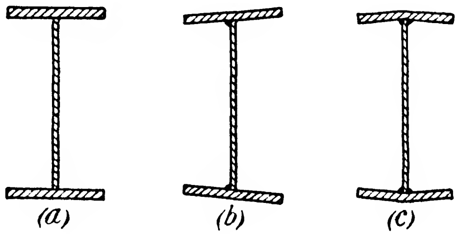


FIG. 98. DEFORMATION OF PLATE GIRDERS DURING WELDING

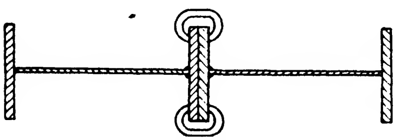


FIG. 99. DEFORMATION OBVIATED BY MAKING PULL OF WELDS COUNTERACT EACH OTHER

little, if any, distortion will be observed. When building heavy girders or frames, particularly from pieces curved or bent to shape, the material is sufficiently rigid in itself to resist distortion and usually no deformation takes place.

Distortion is liable to occur when pieces of different thickness or heat capacity are welded together, owing to the relative displacement of one piece to the other during welding. When a flat is welded by its edge to the surface of a plate as in Fig. 100 (*a*), the heat capacity of the edge of the flat is small compared with that of the plate, and, as welding proceeds, the heat builds up in the flat and runs ahead of the weld more than in the plate, and consequently the edge of the flat is extended more than the plate and the edge of the flat moves forward relative to the plate. As welding proceeds, the flat is fixed in its extended condition and, when the piece cools, locked up stresses remain in the piece and cause distortion (Fig. 100 (*b*)).

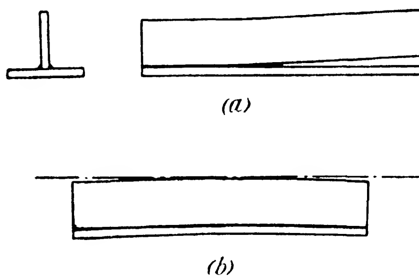


Fig. 100. Deformation of Member with Pieces of Different Heat Capacity

It can be seen that, in so far as the amount of the distortion depends on the extent to which the heat runs ahead of the arc, the distortion will be reduced, and in some cases entirely eliminated, merely by increasing the speed of welding. If it is not, it may be necessary to spread the heat by making the welds in short lengths.

A procedure adopted with success in a work of some difficulty was as follows. The pieces were tacked rigidly together at short intervals to prevent any relative displacement. The joint was marked off into short lengths ($4\frac{1}{2}$ in.) and the welds were made in the sequence indicated in Fig. 101.

The weld was specified as 1–10/9 so that two welds were made with each electrode. All the welds were made inwards

towards a fixed point. Considering the making of the first weld it is seen that the plates are connected by a tack weld at the end of the weld, and as soon as welding is commenced they are connected again by the weld itself. Any movement which might take place between the two plates is restricted to the short length of the weld itself, and in this length the plates themselves are sufficiently strong to resist any movement.

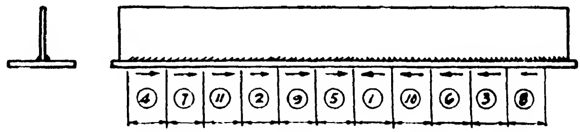


FIG. 101. ARRANGEMENT OF WELDS TO OBVIATE DISTORTION

Although the procedure may appear complicated at first sight it is quite simple in operation and does not affect the speed of welding to any extent. A variation of this method, known as the "back-stepping" method, is indicated in Fig. 102. Each

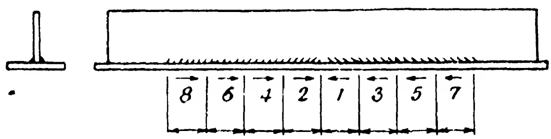


FIG. 102. ALTERNATIVE ARRANGEMENT OF WELDS (BACK-STEPPING) TO OBVIATE DISTORTION

weld is started on cold plate and the welding proceeds inwards to join up with the beginning of the previous run.

Since the size of the deposit influences the heat input and consequently any relative difference of temperature in the two pieces, the smaller the deposit the less likely it is to cause distortion, but a comparatively large run of metal may be deposited from a large gauge electrode without causing distortion if the speed of travel in making the weld is sufficiently fast. When the welds in a structure or a structural unit are properly proportioned for the work they have to do, there is not, as a rule, any serious tendency for distortion to occur.

The following rules will be found to assist in avoiding distortion—

1. If distortion is liable to occur, the work should be designed so that the pull of one weld may be counteracted by the pull of another weld.

2. The plates should be tacked rigidly together at intervals depending on the thickness of the plate (for $\frac{3}{8}$ in. plate about 3–4 ft.) in order to prevent relative displacement of the pieces.

3. The individual runs should be made small in comparison with the thickness of the plate.

4. The speed of travel during welding should be as rapid as possible to prevent the heat running ahead of the arc.

5. The welding may be done in small sections to spread the heat evenly about the work, if necessary using the "back-stepping" method or some variation of it.

6. The shape of the piece should be checked from time to time during welding and, if distortion commences, work should be stopped on the weld causing deformation and welding commenced on the opposing weld to bring the piece back straight.

7. When the arrangement of the structure is such that it is not possible to balance the welding, the welds may be peened after each run to stretch the weld metal and thus to counteract the cooling shrinkage.

APPENDIX

L.C.C. WELDING REGULATIONS

Regulations made under section 9 (2) *of the London Building Act* (*Amendment*) *Act*, 1935, *relating to applications for modification or waiver of building by-laws Nos.* 63, 64, 69, 70, 72, 74, 75, 78, 81, 85, 86, 87, 90, 91, 104 *and* 114, *so as to permit the use of electric* (*metal*) *arc welding instead of riveting, bolting or lapping.*

1. Each application for permission to use electric (metal) arc welding (instead of riveting, bolting or lapping as required and provided for in the building by-laws) should be accompanied by adequate particulars, calculations and plans relating to the character and quality of the welding proposed and to the manner in which it is proposed to be used.

If such application be granted, the Council will prescribe such conditions as it may deem proper to the use of the welding in the manner proposed for that case; and such conditions will apply only in respect of the building to which such permission relates.

The Council will, however, be prepared to consider preliminary applications for approval in principle of the adoption of welding in relation to the construction of a building. If such application is granted it will be necessary for the detailed consent of the Council to be obtained in due course of the methods to be adopted.

2. Structural steel parts for connection by welding should comply with the requirements of the British Standard Specification No. 15—1936.

3. Provision in accordance with the building by-laws should be made for all the effects (e.g. of continuity or rigidity) consequent upon the use of welding instead of riveting, bolting or lapping.

4. Terms relating to electric (metal) arc welding used in applications to the Council for the use of such welding should bear the meanings assigned to them in the British Standard Specification No. 499—1933. The terms used herein bear the same meanings as in that Specification.

5. The applicant should furnish in each case evidence to the satisfaction of the Council as to the strength, ductility and other essential properties of electrodes and of weld metal. Such evidence should be suitable and sufficient to enable the Council to decide whether—and, if so, the conditions under which—the welding proposed may be used in that case.

The applicant shall furnish also in each case evidence to the satisfaction of the Council as to the means proposed for—

(*a*) ensuring that the welding will be executed by competent and reliable operatives;

(*b*) supervising the work of each operative welder during progress; and

(*c*) ensuring that defective work will not be incorporated in the building to which the consent of the Council relates.

6. The Council will determine in each case the maximum permissible stresses, the detail arrangement of connections and such other restrictions as the Council may deem proper for the use of such welding in the manner proposed.

The following table may, however, be taken as a general indication of the probable maximum stresses which will be permitted by the Council—

Classification of stress in welded connections	Maximum permissible stress, in tons per in.^{2}
Tension and compression in butt welds	8
Shearing in butt welds in webs of plate girders and joists	6
Shearing in butt welds other than webs of plate girders and joists	5
Stress in end fillet welds	6
Stress in side fillet welds, diagonal fillet welds and tee fillet welds	5

7. A square butt weld should not be used when the thickness of the parts to be joined exceeds $\frac{3}{16}$ in.

8. When a J or bevel butt weld must be used, the maximum permissible stresses should be reduced to three-fourths of those specified in clause 6.

9. Subject to the provisions of clause 7, any of the forms of butt weld specified in clause (ii), except a square butt weld, may be used provided the parts to be joined be not less than $\frac{3}{16}$ in. in thickness; but the form and dimensions of the weld surfaces should be such as will provide access for the electrode to the surfaces to be welded, and enable the welder to see clearly the work in progress.

10. Steel parts less than $\frac{3}{8}$ in. in thickness should, before butt welding, be separated by a gap not less than $\frac{1}{16}$ in.

Steel parts not less than $\frac{3}{8}$ in. in thickness should, before butt welding, be separated by a gap not less than $\frac{1}{8}$ in.

Provided that in bevel welds the gaps above specified should be not less than $\frac{1}{8}$ in. and $\frac{3}{16}$ in. respectively.

11. A root face (if any) in a butt weld should be not more than $\frac{1}{16}$ in. in width for steel parts not more than $\frac{1}{2}$ in. in thickness, nor more than $\frac{1}{8}$ in. in width for steel parts more than $\frac{1}{2}$ in. in thickness.

In the case of a double V or a double bevel butt weld there should be no root face.

12. The included angle of a V butt weld should be not less than 70° nor more than 100°.

13. In a bevel butt weld, the angle of bevel should be not less than 45° nor more than 50°; and the edges of the steel parts should, before welding, be separated by a gap not less than $\frac{1}{8}$ in. if the parts are less than $\frac{3}{8}$ in. in thickness, and by a gap not less than $\frac{3}{16}$ in. if the parts are not less than $\frac{3}{8}$ in. in thickness.

14. In a U butt weld, the radius at the bottom of the U should be not less than $\frac{1}{8}$ in., and the angle of bevel on each face shall be at least 10°.

15. In a J butt weld, the radius at the bottom of the J

should be not less than $\frac{3}{16}$ in.; and the angle of bevel should be not less than 20° nor more than 30°.

16. Where steel parts of different thicknesses are butt welded, and the surfaces of the steel parts are $\frac{1}{4}$ in. or more out of line, the thicker part should be bevelled so that the slope of the surface from one part to the other is not more steep than one in five (see Fig. 10).

Alternatively, the weld metal should be built up at the junction with the thicker part to a thickness at least 25 per cent greater than the thickness of the thinner part.

17. (*a*) Single V, U, J or bevel butt welds should be reinforced wherever practicable by depositing a run of weld metal on the back of the joint. Where this is not done, the maximum stress in the weld should be (except as provided in paragraph (*b*) of this clause) not more than one-half of the corresponding stress indicated in clause 6.

(*b*) Where it is not practicable to deposit a run of weld metal on the back of the joint, then, provided another steel part is in contact with the back of the joint, and provided also that the steel parts are bevelled to an edge with a gap of at least $\frac{1}{8}$ in. to ensure fusion into the bottom of the V and the steel part at the back of the joint, and provided further that the first run is made with an electrode not larger than No. 8 (S.W.G.), the working stress should not exceed that indicated in clause 6.

18. (*a*) A butt weld should be reinforced so that the thickness at the centre of the weld is at least 10 per cent more than the thickness of the steel parts joined.

(*b*) Where a flush surface is required, the butt weld should be first reinforced as in paragraph (*a*) of this clause, and then dressed flush.

(*c*) Where a butt weld is dressed flush in accordance with paragraph (*b*) of this clause, the working stress in the weld metal should not exceed that specified in clause 6.

19. The throat thickness of a butt weld should be taken as the thickness of the thinner of the steel parts joined.

20. The size of a fillet weld should be specified by the length of the shorter leg (see Fig. 11).

The throat thickness of a fillet weld should be not less than 0·7 of the size (see Fig. 11).

21. The strength of a fillet weld should be calculated on a dimension of 0·7 of the size.

The effective length of a fillet weld (for the purpose of stress calculation) should be deemed to be the overall length of the weld minus twice the weld size.

22. The minimum effective length of a fillet weld required to transmit loading should be not less than 2 in. nor less than six times the size of the weld.

Fillet welds connecting steel parts the surfaces of which form an angle less than 60° or more than 110° should not be relied upon to transmit loading.

23. A side fillet weld is a fillet weld stressed in logitudinal shear—i.e. a fillet weld the axis of which is parallel with the direction of the applied load.

24. An end fillet weld is a fillet weld stressed in transverse shear—i.e. a fillet weld the axis of which is at right angles to the direction of the applied load.

25. A diagonal fillet weld is a fillet weld inclined to the direction of the applied load.

26. A tee fillet weld is a fillet weld joining two steel parts, the end or edge of one part butting on a surface of the other part (see Fig. 11).

27. The actual lengths, sizes and types of welds should be clearly specified on the particulars, calculations and plans to be submitted to the Council. Symbols used should be as specified in the British Standard Specification No. 499—1933.

28. The effective cross section of a weld should be taken as the effective length of the weld multiplied by the throat thickness as specified in clause 19 for butt welds or as specified in clause 20 for fillet welds.

29. The effective section modulus (Z) of a weld or of a group of welds in a plane of a connection should be taken as the

moment of inertia of the effective cross section of the weld or group about the neutral axis of the weld or group divided by the distance between the neutral axis and the edge of the effective cross section farthest from it.

30. The direct stress (f) in fillet or in butt welds of connections stressed in tension, compression or shear should be computed by the following formula (1)—

$$f = P/A \quad . \quad . \quad . \quad . \quad (1)$$

where P is the load to be transmitted by the connection, and A is the effective sectional area of the weld or welds transmitting such load.

31. The stress in the weld or welds of a connection due to bending should be computed by the following formula (2)—

$$f_b = M/Z \quad . \quad . \quad . \quad . \quad (2)$$

where f_b is the stress due to bending, M is the bending moment transmitted by the connection and Z is the effective section modulus of the weld or welds.

32. When the weld or welds in a connection are subjected to the action of bending combined with direct stress due to shear, tension or compression, the direct stress should be computed by formula (1), the stress due to bending should be computed by formula (2), and the resultant direct stress f_r should be determined therefrom. The resultant tensile or compressive stress f_r should not exceed the maximum permissible stress specified for tension or compression in clause 6.

33. The arrangement of welds at a joint should be such that uncertainty as to the distribution of stress is avoided as far as practicable. Where an eccentric connection cannot be avoided the bending effect should be computed, and proper allowance made.

34. Members and connections should be so designed that component parts may be readily assembled and securely held in place by means of clamps or other devices. The welds should be so located as to be readily accessible for welding, inspection, painting and maintenance.

35. Connections for bracing members, of which the sections are not determined by calculated stresses, should be designed to develop the effective strength of the member.

36. In all cases where welded joints may be exposed to weather, the joining edges of the contact surfaces should be sealed by welding, or the parts should be effectively connected by welding so that the contact surfaces are securely held in contact to prevent the entry of moisture.

37. (*a*) Intermittent fillet welds may be used when continuous welds are not required for strength; intermittent butt welds should not be used.

(*b*) The longitudinal space between intermittent fillet welds should not exceed 16 times the thickness of the thinner plate in tension members or 12 times the thickness of the thinner plate in compression members.

38. For the purpose of extending the length of fillet welds within the space occupied by a joint, slots or holes may be made through one or more of the plates forming the joint; the slot or hole should not be filled with weld metal nor partially filled in such a manner as to form a direct weld metal connection between opposite sides of the slot.

The dimensions of the slot or hole should comply with the following limits in terms of the thickness of the steel part in which the slot or hole is formed—

(*a*) Width to be not less than twice the thickness, with a minimum of 1 in.

(*b*) Corners to be rounded with a radius not less than the thickness, with a minimum of ½ in.

(*c*) Distance from the edge of the member of slot or hole to be not less than twice the thickness.

39. Combination of side and end fillet welds should be used in preference to side or end fillet welds alone.

40. If side fillet welds are used in end connections, the length of each side fillet weld should be not less than the distance between them. The side fillet welds may be either at the edges of the members or in slots or holes.

41. In end connections a single end fillet weld should not be used without side fillet welds. Where two or more end fillet welds are used without side fillet welds, the end of each fillet weld should, wherever possible, be returned as a side fillet weld for a length of at least 1 in., and in this case the full length of the end weld may be used for the purpose of calculating its strength, the return welds being disregarded.

42. In lap joints between plates, the minimum amount of lap should be at least four times the thickness of the thinner plate.

43. In built-up members in which plates are connected by intermittent fillet welds, continuous side fillet welds should be used at the ends for a length not less than the width of the plate connected.

44. In built-up members the unsupported width of a plate between the welds connecting it should not exceed 40 times the thickness of the plate.

45. Welds joining the flange to the web of a plate girder should be so designed that they will be capable of transmitting the shearing forces between the flange and the web without exceeding the maximum permissible stresses.

46. The shear stress in the splices of plate girder webs may be assumed to be uniformly distributed over the whole depth of the web, and the stresses in the welds should not exceed the permissible working stress specified in clause 6.

47. For plate girders with unstiffened webs, the thickness of the web plate should be not less than one sixtieth of its depth.

48. Column joints should be machined properly to fit. For column joints in which the resultant stress due to all loads and bending moments is wholly compressive, the welds should be sufficient to retain the members accurately in place. For column joints in which bending stresses can produce tension the welds should be designed to resist such bending.

49. The connections between beams and columns should be designed so that—

(*a*) The connection is virtually unable to afford restraint to the free deflection of the beam and in such cases the beam should be treated as freely supported; or

(*b*) the connection is virtually rigid and designed to be capable of transmitting the full moments and shear which are set up at the joint. In this case the design should make allowance for the stresses set up in other parts of the structure.

The several parts of the welded joint should be designed to transmit the forces set up, and the following methods of construction should be avoided—

(*a*) Connection of beams to columns by means of welds on the web of the beam only.

(*b*) Connections of beams to columns by means of welds on the flanges of the beam only without brackets or stools to transmit shear.

Information for the Guidance of Applicants

The Council may include in the conditions upon which a waiver or modification is granted the following requirements—

(i) Electrodes for welding should comply with the requirements for Class A electrodes in the British Standard Specification No. 639—1935.

(ii) Butt welds should be made in one of the following forms—

(*a*) Square butt joint, as shown in Fig. 1;
(*b*) Single V butt joint, as shown in Fig. 2;
(*c*) Double V butt, as shown in Fig. 3;
(*d*) Single U butt joint, as shown in Fig. 4;
(*e*) Double U butt joint, as shown in Fig. 5;
(*f*) Single J butt joint, as shown in Fig. 6;
(*g*) Double J butt joint, as shown in Fig. 7;
(*h*) Single bevel butt joint, as shown in Fig. 8;
(*i*) Double bevel butt joint, as shown in Fig. 9.

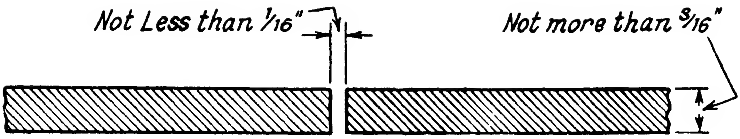


Fig. 1. Square Butt Weld

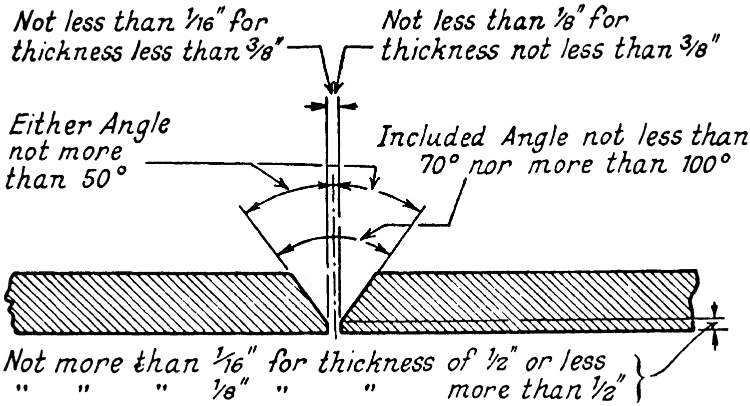


Fig. 2. Single V Butt Weld

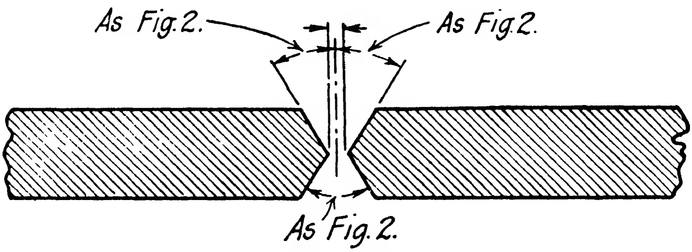


Fig. 3. Double V Butt Weld

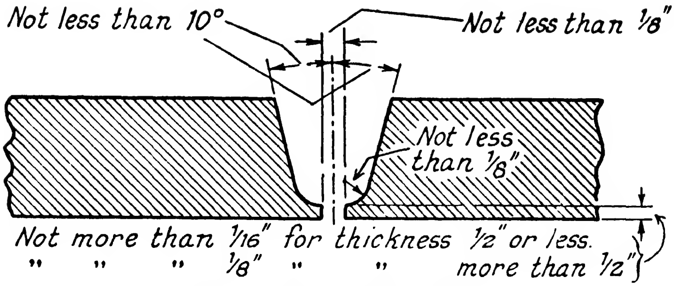


FIG. 4. SINGLE U BUTT WELD

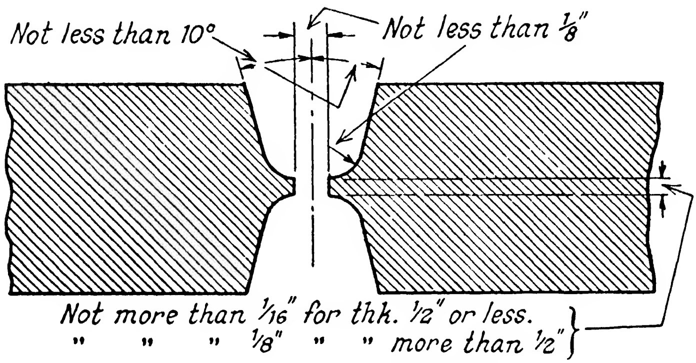


FIG. 5. DOUBLE U BUTT WELD

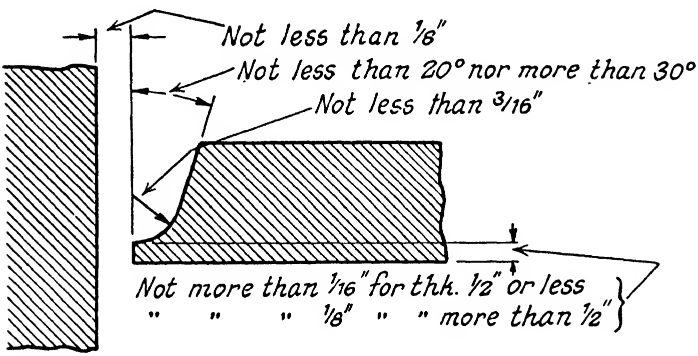


FIG. 6. SINGLE J BUTT WELD

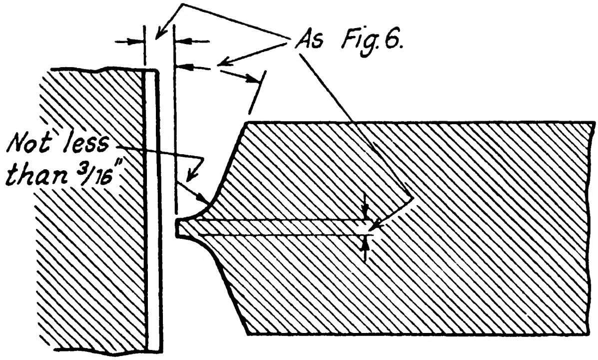


Fig. 7. Double J Butt Weld

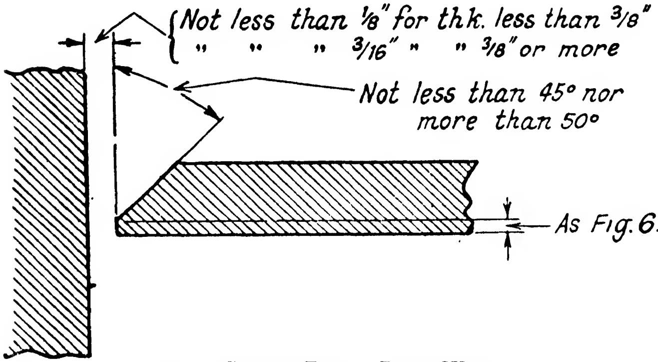


Fig. 8. Single Bevel Butt Weld

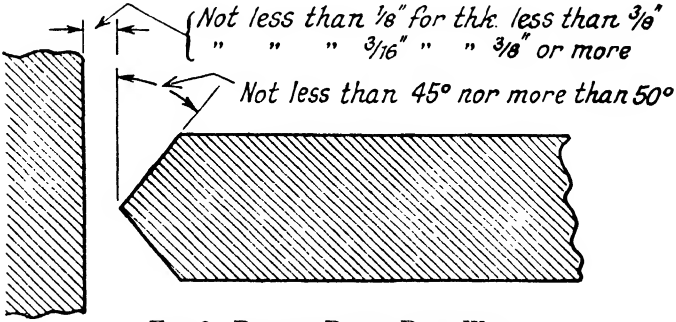


Fig. 9. Double Bevel Butt Weld

(iii) The applicant should furnish in each case, to the satisfaction of the district surveyor, as and when he may require, evidence that welding used or to be used is in accordance with the conditions prescribed by the Council in that case.

(iv) The surfaces to be welded and the surrounding material for a distance of at least half an inch should be freed from scale

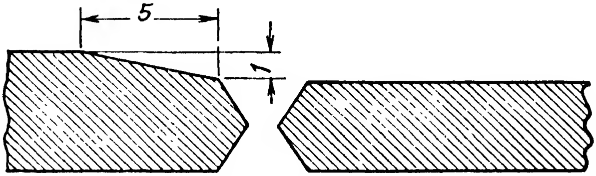


FIG. 10. THICKER PLATE BEVELLED TO THICKNESS OF THINNER PLATE

and cleaned so as to remove dirt, grease, paint, heavy rust or other surface deposit, wire brushing being used if necessary. A coating of linseed oil applied for the purpose of preventing corrosion may be disregarded.

(v) Fusion faces which require to be cut to a special form or shape may be cut by shearing, clipping, machining or by a gas

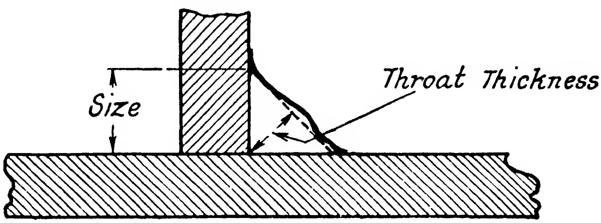


FIG. 11. THROAT THICKNESS

cutting machine. Hand cutting by gas may be substituted for machine cutting only when cutting by machine is, in the opinion of this district surveyor, impracticable, and should be so carried out that the effect of cutting is uniform.

If the prepared fusion face is irregular, it should be dressed, to the satisfaction of the district surveyor, by chipping, filing or grinding.

(vi) The pieces to be welded should be securely held in their correct relative positions during welding.

The welding sequence adopted should be such that distortion is reduced to a minimum.

(vii) The deposition of the weld metal should be carried out so as to ensure that—

(*a*) Welds will be of good clean metal, deposited by a process which will ensure uniformity and continuity of the weld; and

(*b*) The surfaces of the weld will have an even contour and regular finish indicating proper fusion with the parent metal.

Welds showing cavities, or in which the weld metal tends to fall over on the parent metal without proper fusion, should be cut out and rewelded.

Care should be taken to avoid undercutting; and where serious undercutting occurs, the reduction at that point should be made good by an additional run of weld metal if the district surveyor so requires.

All slag should be removed after making each run, and for this purpose light hammering followed by wire brushing (or other methods which will not disturb the weld) may be used.

The electric current used in making welds should be within the range defined by the manufacturer of the electrodes used.

(viii) Finished welds and adjacent parts should be coated with clean linseed oil immediately all slag has been removed.

The welds and adjacent parts should not be painted until the district surveyor has had two working days in which to approve them.

(ix) Welders should be provided with such staging as will enable them properly to perform the welding operations. For site welding, shelter should be provided to protect welders and the parts to be welded from the weather.

(x) Adequate steps shall be taken to ensure that the work is of the highest quality and thoroughly reliable and that all work is done under competent and skilled supervision.

Note. The above information is intended only as a general guide for applicants as the Council will deal with each application on its merits.

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